IS 4880-1 (1987): Code of practice for design of tunnels conveying water, Part 1: General design [WRD 14: Water Conductor Systems]

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Indian Standard

CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART 1 GENERAL DESIGN

(First Revision)

First Reprint APRIL 1996

UDC 624·191·1 : 624·196

BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

Gr 2

April 1988

AMENDMENT NO. 1 MARCH 2004
TO
IS 4880 (PART 1) : 1987 CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART 1 GENERAL DESIGN

(First Revision)

[*Page* 3, *clause* **3.1**(j)] — Insert the following at the end:

k) Brittleness test ;

m) Stever's 'J' value tests; and

n) Abrasion test.

(*Page* 3, *clause* **4.1**) — Insert the following at the end:

'f) *Pore Pressure Observations* — Pore pressure meter for monitoring the pore water pressure around the tunnels.'

(*Page* 3, *clause* **4.4**) — Insert the following new clause after **4.4**:

'**4.5** Numerical tools should be used to carry out stress analysis using the laboratory and *in-situ* test results to predict the likely stress pattern and deformation around tunnels after excavation. The same shall be compared with the instrumentation observations on the tunnels and thereby analysis should be refined as construction progresses.'

(WRD 14)

Reprography Unit, BIS, New Delhi, India

Indian Standard

CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART 1 GENERAL DESIGN

(First Revision)

0. FOREWORD

0.1 This Indian Standard (Part 1) (First Revision) was adopted by the Bureau of Indian Standards on 30 October 1987, after the draft finalized by the Water Conductor Systems Sectional Committee had been approved by the Civil Engineering Divison Council.

0.2 For the alignment of tunnels and designs of tunnel supports and lining, the nature of soft or hard strata and its formation plays a vital role. It is necessary to know the general topography, the geology of the area, state of stress and other mechanical properties of the strata. For this certain topographical and geological investigations, *in-situ* and laboratory test, and observations are necessary. For certain locations where difficult working conditions are anticipated, more detailed investigations may be undertaken.

0.3 This standard has been published in various parts. Other parts of this standard are as follows:

- Part 2 Geometric design
- Part 3 Hydraulic design
- Part 4 Structural design of concrete lining in rock
- Part 5 Structural design of concrete lining in soft strata and soils
- Part 6 Tunnel supports
- Part 7 Structural design of steel lining

0.4 This standard was first published in 1975. The present revision of the standard has been taken up in the light of experience gained during the last few years in the use of this standard. In this revision, the clauses on '*in-situ* rock and tests' and 'instrumentation' have been modified to introduce the modern rock mass classification.

1. SCOPE

1.1 This standard (Part 1) covers the general requirements, like various types of investigations, tests and instrumentation of tunnel generally required for planning and designing of pressure tunnel section and supports.

2. INVESTIGATIONS

2.1 General — Records of any existing tunnels and other excavations in the vicinity including any information regarding old mine workings or old wells, should be sought and studied. Information should also be sought in historic records concerning flooding, avalanches, landslips, earthquakes, etc.

2.2 Topographical Surveys — Surveys for preparation of plans and aligning the tunnel should be carried out covering the area of tunnel alignment, after establishing adequate number of temporary bench marks with reference to the G.T.S. bench mark available in the vicinity. The survey shall be carried out in accordance with the provisions contained in IS : 5878 (Part 1)-1971*. Where movements along faults are suspected, local network of survey monuments shall be laid and observations made during construction period as also later during operation.

2.2.1 Preliminary investigations for aligning the tunnel should be carried out on available 1 : 50 000 Survey of India Topo Sheets. Once the general feasibility of the tunnel is established, detailed strip topographic maps along the tunnel alignment should be prepared to a scale 1 : 10 000 with 5 m contour interval. Width of the strip may be fixed on the basis of investigations, which shall be carried out more intensely at locations where certain local geologically adverse features like major shears, thrusts,

*Code of practice for construction of tunnels: Part 1 Precision survey and setting out.

faults synclines, etc, exist or where exposed rock is encountered and where the rock cover is less than the internal water pressure at that location. The strip width shall be commensurate with the internal water pressure on either side of alignment and also up to contours corresponding to tunnel grade indicating location of adits where necessary. At portal faces, the contour interval should be reduced to 2 m.

2.2.2 Wherever possible, aerial (photographic) survey should be carried out and the stereoptic coverage should extend for at least 3 km on either side of the possible foreseen limits of the tunnel alignment. This would facilitate to pinpoint those areas that require surface and subsurface investigations for a detailed assessment. If infra-red aerial photography is used, it would facilitate to delineate hot water bearing zones in bed rock.

2.3 Geological Investigations — Geological investigations should be carried out with sophisticated instruments, some of which are listed in **4.1**. If the area has been aerially photographed, such data should be studied.

2.3.1 The geological investigations should be carried out to determine:

a) Origin and type of rock along the alignment and study of regional geological maps of the area, if available;

b) Geological section along the tunnel alignment giving rock types and their disposition; location and attitude of all structural features of rock such as faults, thrusts, joints, dips, strikes and other geological features including pattern, extent and contents of fissures; presence of water in small or large quantities and their probable pressure at tunnel grade, etc;

c) Any geological feature which may affect the magnitude of rock pressure to be anticipated along the proposed alignment;

d) Cover on the tunnel, position of subsurface rock and overburden contacts;

e) Physical, mechanical and strength properties of rock to determine supporting arrangements and also resistance to driving tunnel through rock (if tunnelling with a mole is proposed); and

f) Hydrological data and information regarding location, type and volume of water and injurious or troublesome gases contained in subsurface strata around tunnel grade.

2.3.2 The geological data should be developed through a comprehensive geological investigation which includes:

a) *Detailed geological mapping* — Detailed geological mapping to know the rock formations, locations and altitude of structural features such as folds, faults, joint pattern, etc, to plan drill holes;

b) *Subsurface exploration* — Few cored bore holes should be taken at suitable locations along the alignment of tunnel as suggested by geologist. The number of bore holes depends upon the length of tunnel, rock cover over tunnel grade, number of adits available and geological features likely to be met with. However, the minimum number of bore holes as adjudged to be necessary by an experienced engineering geologist in consultation with design engineers should be provided. For proper determination of rock quality designation (RQD) (*see* **3.2.3**), the bore holes should be drilled with *NX* size and larger size only and not that *BX* or smaller sizes. The core samples of each bore hole shall be preserved and logged by an engineering geologist. Bore holes shall avoid, as far as possible, intercepting tunnel bore, particularly in water bearing strata, and shall be properly backfilled preferably with concrete;

c) *Geophysical investigations* — This type of investigation is helpful in establishing the rock-soil boundary, in delineating fault and shear zones, other geological structures and similar phenomenon. This investigation is also used in evaluating rock mass quality by determining *in-situ* modulus of elasticity;

d) *Television investigation of bore holes* — If possible, the walls of bore holes may be examined by television bore hole cameras. This method facilitates in studying the depth of altered rock, location and determination of the altitude and character of shear zones, joints fractures, foliations and bedding planes, assessment of rock condition above and below the water table, identification of rock types and other visually detectable geological characteristics of in-place rock prior to excavation;

e) *Exploration drifts* — Drifts should be provided at portals or at adit points. These are most accurate means of determining the geological conditions in tunnelling and for conducting *in-situ* rock tests.

2.3.3 Geological investigations should be continued during construction not only in the interest of checking design data but also for ascertaining the tunnelling methods and predicting tunnel conditions ahead of tunnel face to minimize surprises.

3. TESTS

3.1 Laboratory Tests — The core samples collected from the bore holes shall be classified and specimen from each group shall be tested to determine the following physical properties:

a) Specific gravity,

b) Modulus of elasticity (static and/or dynamic),

c) Poisson's ratio,

d) Tensile strength,

e) Compressive strength (dry and wet),

f) Triaxial shear strength,

g) Hardness of rock,

h) Swelling index (in case of soft argillaceous rocks), and

j) Porosity, grain size and cementing material for sand stones and similar rocks.

3.2 *In-situ* Rock Tests

3.2.1 The data obtained from field and laboratory tests shall be substantiated by *in-situ* rock tests. When a cavity is formed in the rock mass, the *in-situ* rock stresses are altered for some distance around the opening. *In-situ* rock tests are carried out to evaluate:

a) *In-situ* rock characteristics like shear strength parameters (*C* and *l*), compressive strength and deformation modulus preferably by Goodman Jack;

b) Deformation of rock around opening;

c) Rock load on supports — temporary and permanent; and

d) The tests shall be carried out in two directions at right angles to each other in case of laminated rock structures—one parallel to and the other at right angles to the dip and strike of rock. Plastic fields shall be determined by repeated loading and unloading tests.

3.2.2 The information obtained from **3.2.1** is required for providing supporting system in tunnel and design lining. These are to be obtained by installing instruments described in **4.1**.

3.2.3 From the bore hole logs, rock quality designation (RQD) should be determined. Geotechnical and geological data should be collected with a view to enable modern rock mass classification [*see* IS : 11315 (Part 11)-1985*].

4. INSTRUMENTATION

4.1 Systematic instrumentation is to be done in all major tunnels under construction to monitor the behaviour of supports and the rock. Such a study may be started from the very start of the tunnel. The instruments should be installed at the time of installation of the supports. The following may be done. The instrumented section should be so dispersed as to cover statistically differing rock conditions:

a) *Closure Observations* — Tunnel closure should be observed at random interval throughout the length of the tunnel;

b) *Bore-Hole Extensometer* — Multipoint bore-hole extensometer should be used to know the deformation in the rock around the tunnel opening. The observations will help in ascertaining the shape and size of the plastic (broken) zone. A minimum of three, that is, one horizontal, one vertical and one at 45° to the horizontal per section should be used;

c) *Load Observations* — Rock load coming on the steel supports should be monitored by installing load cells on ribs. A minimum of three per section should be used;

d) *Contact Pressure Observations* — Pressure cells should be placed at the intervals of the supports and the rock surface to measure rock pressure and internal water pressure. The pressure cells should not be placed at preferably less than 60°; and

e) *Strain Observations* — Should be done by embedding strain meters in concrete lining for the measurement of stress in the lining.

4.2 The instruments mentioned in **4.1** may be provided at more than three sections or at the typical representative reaches met with while excavating. The range of instruments to be installed depends upon rock cover, internal pressure and geological features and properties of rock mass and should be fixed after due analysis. Instrumentation may be done in the drifts which are made during investigation so that the data can be made available for design of supports and lining during execution of the work.

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*Method for the quantitative descriptions of discontinuities in rock masses: Part 11 Core recovery and rock quality.

4.3 Suitable instruments may be used for construction and post-construction stages.

4.4 The observations shall be taken in accordance with the format and frequency suggested by the experts.

5. GENERAL DESIGN

5.1 Investigations as detailed in **2**, **3** and **4** can be used in general designing of the tunnel which can be proceeded with as laid down in the following six parts of this code:

- Part 2 Geometric design,
- Part 3 Hydraulic design,
- Part 4 Structural design of concrete lining in rock,
- Part 5 Structural design of concrete lining in soft strata and soils,
- Part 6 Tunnel supports, and
- Part 7 Structural design of steel lining.

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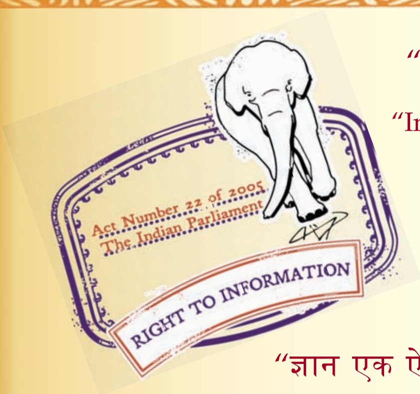

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“Knowledge is such a treasure which cannot be stolen”

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(Reaffirmed 2000)

Indian Standard

CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART II GEOMETRIC DESIGN

(First Revision)

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NEW DELHI 110002

Gr 3 *September* 1976

Indian Standard

CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART II GEOMETRIC DESIGN

(First Revision)

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Indian Standard

CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART II GEOMETRIC DESIGN

(First Revision)

0. FOREWORD

0.1 This Indian Standard (Part II) (First Revision) was adopted by the Indian Standards Institution on 24 July 1976, after the draft finalized by the Water Conductor Systems Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 This Indian Standard was first published in 1968. Its revision was taken up with a view to keeping abreast with the technological developments that have taken place in the field of tunnel design and construction. This revision incorporates modified Fig. 1 which more clearly illustrates *A*-line and *B*-line. A new geometric shape, egglipse, has also been added to the list of sections recommended for adoption for tunnels. The details for drawing the egglipse curve have been included as Appendix A.

0.3 Tunnels are generally used for conducting water through high ground or mountains, in rugged terrain where the cost of a surface line is excessive and elsewhere as convenience and economy dictate.

0.4 This standard has been published in parts. Other parts of the standard are as follows:

(Part I)-1975	General design
(Part III)-1976	Hydraulic design (*first revision*)
(Part IV)-1971	Structural design of concrete lining in rock
(Part V)-1972	Structural design of concrete lining in soft strata and soils
(Part VI)-1971	Tunnel supports
(Part VII)-1975	Structural design of steel lining

0.4.1 This part (Part II) lays down only general guidance in regard to the shape of various sections generally used for tunnels. However, for particular project the judgement of the designer is required for making a final choice of a section considering the prevailing site conditions, since no general recommendations can be made to fit in each and every individual case.

0.5 This code of practice represents a standard of good practice and therefore, takes the form of recommendations.

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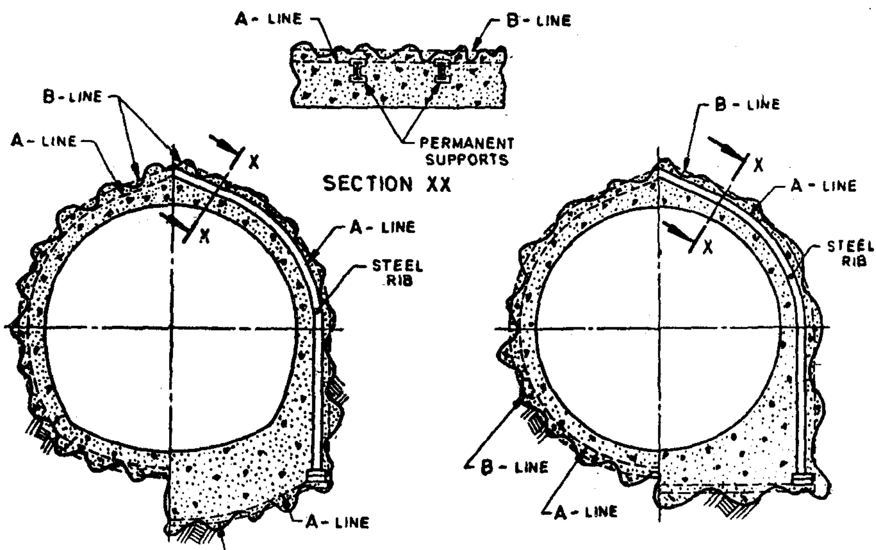
1. SCOPE

1.1 This standard (Part II) lays down general requirements and criteria for geometric design of tunnels conveying water under pressure or under free-flow conditions. This standard does not, however, cover the geometric design of other tunnel structures.

2. TERMINOLOGY

2.0 For the purpose of this standard, the following definitions shall apply.

2.1 Minimum Excavation Line (*A*-Line) — It is the line within which no unexcavated material of any kind and no supports other than permanent structural steel supports shall be permitted to remain (*see* Fig. 1).



1A HORSE SHOE SECTION WITH AND WITHOUT SUPPORT

1B CIRCULAR SECTION WITH AND WITHOUT SUPPORT

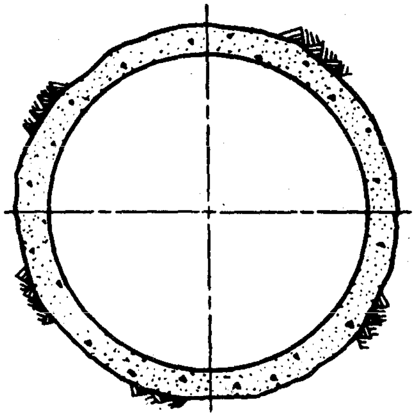
FIG. 1 TYPICAL SECTION OF CONCRETE-LINED TUNNELS SHOWING *A*- AND *B*-LINES

2.2 Pay Line (*B*-Line) — It is an assumed line (beyond *A*-line) to which payment of excavation is made whether the actual excavation falls inside or outside it (*see* Fig. 1). Sometimes *B*-line may merge with *A*-line. It is a common practice to adopt *B*-line for payment for concrete lining.

3. SHAPES

3.1 The following shapes are generally used for tunnel cross sections:

a) Circular section (*see* Fig. 2),
b) D Section (*see* Fig. 3),
c) Horse-shoe section (*see* Fig. 4),
d) Modified horse-shoe section (*see* Fig. 5),
e) Egg shaped section (*see* Fig. 6), and
f) Egglipse section (*see* Fig. 7).



NOTE — For tunnels excavated to horse-shoe section and concreted to circular section, *see* Fig. 1.

FIG. 2 CIRCULAR SECTION

4. GEOMETRIC DESIGN

4.1 Cross section of a tunnel depends on the following factors:

a) Geological,
b) Hydraulic,
c) Structural, and
d) Functional.

NOTE — It is not uncommon that the sections get modified during the course of construction.

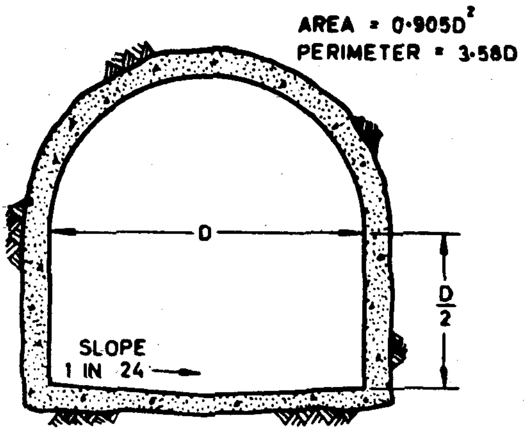


FIG. 3 D SECTION

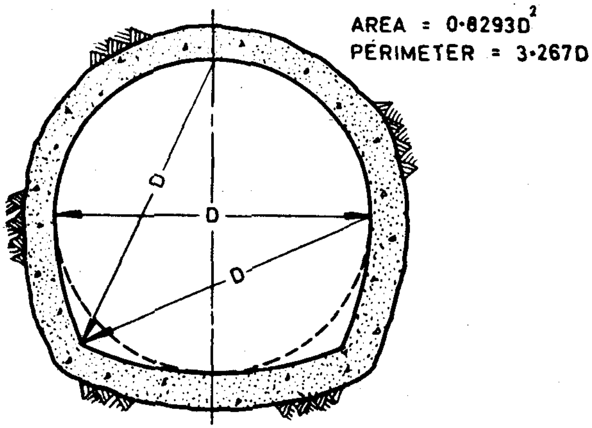
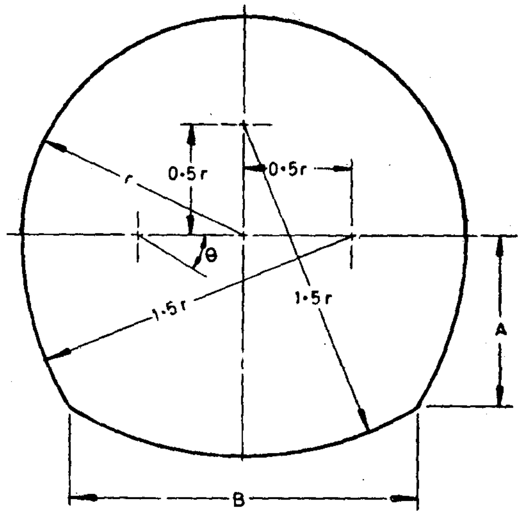


FIG. 4 HORSE-SHOE SECTION

4.1.1 *Circular Section* — The circular section is most suitable from structural considerations. However, it is difficult for excavation, particularly where cross-sectional area is small. For tunnels which are likely to have to resist heavy inward or outward radial pressures, it is desirable to adopt a circular section. In case where the tunnel is subjected to high internal pressure, but does not have good quality of rock and/or adequate rock cover around it, circular section is considered to be the most suitable.



$r = 0{\cdot}987\ 580\ R$

where

R = Radius of Hydraulically Equivalent Circle

Area of Section	$= 3{\cdot}253\ 572\ r^2$
Perimeter of Section	$= 6{\cdot}426\ 334\ r$
Hydraulic Radius	$= 0{\cdot}506\ 287\ r$
A	$= 0{\cdot}780\ 776\ r$
B	$= 1{\cdot}561\ 553\ r$
θ	$= 31°\ 22'\ 01''$

FIG. 5 MODIFIED HORSE-SHOE SECTION

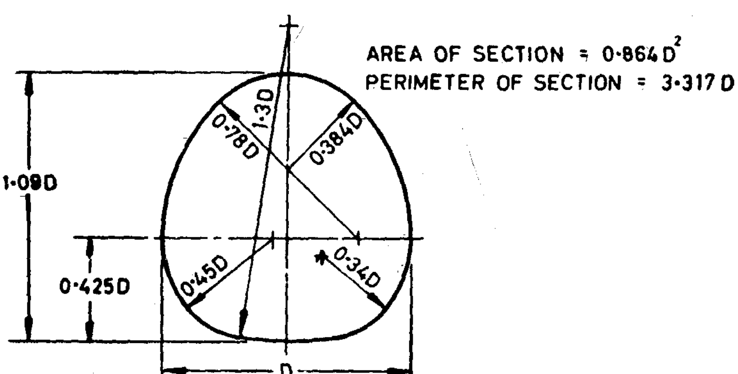


FIG. 6 EGG SHAPED SECTION

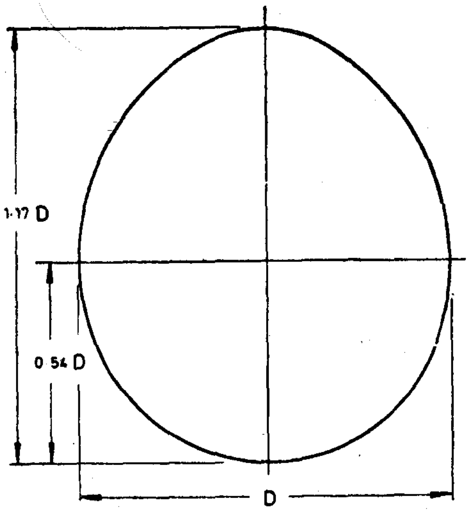


FIG. 7 TYPICAL EGGLIPSE SECTION

4.1.2 *D Section* — *D* section would be found suitable in tunnels located in massive igneous, hard, compacted, metamorphic and good quality sedimentary rocks where the external pressures due to water or unsound strata upon the lining is slight and also where the lining is not required to be designed against internal pressure. The principal advantages of this section over horse-shoe section (*see* **4.1.3**) are the added width of the invert which gives more working floor space in the heading during driving and the flatter invert which helps to eliminate the tendency of wet concrete to slump and draw away from the tunnel sides after it has been screeded.

4.1.3 *Horse-Shoe and Modified Horse-Shoe Sections* — These sections are a compromise between circular and *D* sections. These sections are strong in their resistance to external pressures. Quality of rock and adequate rock cover in terms of the internal pressure to which the tunnel is subjected govern the use of these sections. Modified horse-shoe section offers the advantage of flat base for constructional ease and change over to circular section with minimum additional expenditure in reaches of inadequate rock cover and poor rock formations.

4.1.4 *Egg Shaped and Egglipse Sections* — Where the rock is stratified, soft and very closely laminated (as laminated sand stones, slates, micaceous schists, etc) and where the external pressures and tensile forces

in the crown are likely to be high so as to cause serious rock falls, egg shaped and egglipse sections should be considered. In the case of these sections there is not much velocity reduction with reduction in discharge. Therefore, these sections afford advantage in cases of sewage tunnels and tunnels carrying sediments. Egglipse has advantage over egg shaped section as it has a smoother curvature and is hydraulically more efficient. Details for drawing egglipse curve are given in Appendix A.

4.1.5 *Other Sections* — In addition to the sections mentioned in **4.1.1** to **4.1.4** there may be other composite geometrical sections which may be adopted particularly for tunnels which are free flowing and often only partly lined. If characteristics of a rock formation are fairly well known it may be possible to evolve a section which is likely to fit the shape in which the rock will break naturally. Thus, while a horse-shoe or *D* section is fairly easy to obtain in some formations there are others where the tunnel crown tends to break into a form more nearly square, and if there is no risk of heavy external pressure upon the lining or if the tunnel is to be unlined there is no reason why the designed cross section should not be made to suit the characteristics of the rock.

4.1.6 The typical geometry of both *A*- and *B*-lines for some sections are shown in Fig. 1 and the distance between *A*- and *B*-lines depends on the nature and geology of rock and method of tunnelling.

APPENDIX A

(*Clauses* 0.2 *and* 4.1.4)

DETAILS FOR DRAWING EGGLIPSE CURVE

A-1. GOVERNING RULE

A-1.1 F_1, F_2 and F_3 are the focal points (*see* Fig. 8) of the egglipse. The radii F_1P, F_2P and F_3P are designated as r_1, r_2 and r_3 respectively. The governing rule for any point P on the egglipse is

$$r_1 + r_2 + r_3 = K \qquad \text{..............(1)}$$

where

K is a constant.

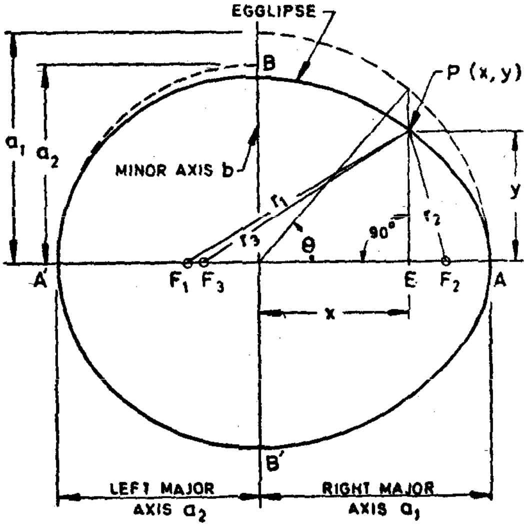


FIG. 8 DETAIL OF EGGLIPSE CURVE

A-2. BASIC EQUATIONS

A-2.1 The basic equations for the egglipse are

$$x = a \cos \theta \qquad \text{.............. (2)}$$

$$y = a \sin \theta - \frac{a - \dfrac{a\,b}{(a^2 \sin^2 \theta + b^2 \cos^2 \theta)^{1/2}}}{\sin \theta} \qquad \text{.............. (3)}$$

where

a is the major axis, and

b is the minor axis.

NOTE — In equations (2) and (3), use a for a_1 for right side curve and use a for a_2 for left side curve, a_1 and a_2 are the right major axis and left major axis respectively.

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CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART III HYDRAULIC DESIGN

(First Revision)

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CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER
PART III HYDRAULIC DESIGN
(First Revision)

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Indian Standard
CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER
PART III HYDRAULIC DESIGN
(First Revision)

0. FOREWORD

0.1 This Indian Standard (Part III) (First Revision) was adopted by the Indian Standards Institution on 24 July 1976, after the draft finalized by the Water Conductor Systems Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 The 'Indian Standard Code of practice for design of tunnels conveying water : Part III Hydraulic design' was first published in 1968. This revision has been taken up with a view to keeping abreast with the technological developments that have taken place in the field of tunnel design and construction. With the confidence gained in the construction of a large number of tunnels and the availability of concretes of higher strengths in the country, the provisions of the code have been recommended for adoption for tunnels carrying water at velocities up to 8 m/s without need for model studies. In keeping with the practice, provision for limiting instant velocity during surge oscillations has been deleted.

0.3 This standard has been published in parts. Other parts of the standard are as follows:

Part I-1975	General design
Part II-1976	Geometric design (*first revision*)
Part IV-1971	Structural design of concrete lining in rock
Part V-1972	Structural design of concrete lining in soft strata and soil
Part VI-1971	Tunnel supports
Part VII-1975	Structural design of steel lining

0.3.1 This part covers recommendations in regard to the hydraulic design of tunnels conveying water. These recommendations may be used for tunnels carrying water at velocities up to 8 m/s. For tunnels carrying water at velocities more than 8 m/s the design based on these recommendations may have to be corroborated by hydraulic model studies.

0.4 This code of practice represents standard of good practice and, therefore, takes the form of recommendation.

0.5 In the formulation of this standard due weightage has been given to international co-ordination among the standards and practices prevailing in different countries in addition to relating it to the practices in the field in this country. This has been met by referring to various publications including the following:

United States of America. Department of the Interior and Bureau of Reclamation. Design of small dams. Government Printing Office, Washington.

United States of America. Department of the Interior and Bureau of Reclamation. Engineering monograph No. 7, friction factors for large conduits flowing full. Government Printing Office, Washington.

Brown (JG), Ed. Hydro-Electric Engineering Practice, Vol I. Blackie & Son Ltd, Glasgow (by permission of the publisher).

1. SCOPE

1.1 This standard (Part III) covers the hydraulic design of tunnels conveying water under pressure or under free flow conditions. This does not, however, cover the hydraulic design of other tunnel structures.

2. GENERAL CONSIDERATIONS

2.1 General — For the hydraulic design, in most cases hydraulic gradient shall be required. However, in addition to hydraulic gradient in certain locations, energy gradient, principles of momentum, transient conditions like water hammer, surges, etc, shall have to be considered. Where air is likely to be entrained because of high velocities, its effect due to bulking should be considered. Due consideration shall be given to maximum and minimum levels at the head and tail end.

2.1.1 The factors which combine to determine the nature of flow in a tunnel include such variables as pressure head, slope, size, shape, length, surface roughness of the tunnel, and the inlet and outlet shapes. The combined effect of these factors determines the location of control which in turn determines the discharge characteristics of the tunnel. In case of free flowing tunnels proper aeration shall be ensured. The tunnel shall be so designed that pulsating conditions are minimised. In the calculation of flow, expected variations in the friction factor shall be considered.

2.2 Obligatory Levels of Tunnel — In case of a pressure tunnel the depth of intake shall be such that no air is sucked in under any condition. The location of outlet of a tunnel shall be such that the entry of air would not adversely affect tunnel operation and safety provided that sufficient precautions for preventing air locks are taken (*see* **6**).

2.2.1 All tunnels should preferably have a positive gradient in the direction of flow, since they may have to be emptied and drained from time to time

for the purpose of inspection and maintenance. However, it may be borne in mind that in a well designed and constructed tunnel there would be only a little need of maintenance. Gradients and depth shall be such that under fluctuating conditions, including transient conditions, there shall be no possibility of air locks.

2.3 Cross Section — The geometric design of various sections usually adopted for tunnels is covered in IS : 4880 (Part II)-1976*.

2.3.1 Area of cross section of a tunnel shall be of sufficient size to carry the maximum required flow on the head available and in addition shall conform to construction requirements.

2.3.1.1 Tunnel dimensions and shape should be decided on the basis of economic studies so as to obtain a most economical section. The following should be taken into account:

a) Velocity requirements,
b) Loss due to tunnel friction,
c) Interest charges on capital cost of tunnel,
d) Annual maintenance charges,
e) Whether lined or unlined, and
f) Cost of gates and their hoists.

2.3.1.2 The tunnel diameter determined as a result of economic studies should be examined from practical considerations, such as space requirements for the excavating equipment and the section may be modified if necessary, based on the above considerations. A minimum height of 2 m is necessary. For mechanized handling of excavated material a minimum section of $2 \cdot 5 \times 2 \cdot 5$ m is required.

NOTE — In sound rock the unit cost of excavation decreases as the diameter increases to a point that permits the use of full sized shovel equipment, say up to 10 m in diameter. In weak rock the unit cost may increase as the size increases owing to extra cost of supports.

2.4 Cavitation — Design shall be such that negative pressures are avoided. To make sure that cavitation is avoided and to allow for uncertainties, the residual positive pressure shall not be less than 3 m of water head in concrete lined tunnels.

2.4.1 The recommended limiting sub-atmospheric pressures, based on probable minimum atmospheric pressures at different elevations above sea level, are indicated in Fig. 1.

NOTE — In locations which are susceptible to effects of cavitation such as downstream of gate slot, where there is a change of grade in high velocity flow, etc, steel lining may be considered.

3. TRANSITION SHAPES

3.1 From the tunnel section, either entry into or exit from the tunnel requires transition to reduce the head losses to a minimum and to avoid cavitation. The length and shape of the transition depends upon the velocity and flow

*Code of practice for design of tunnels conveying water: Part II Geometric design (*first revision*).

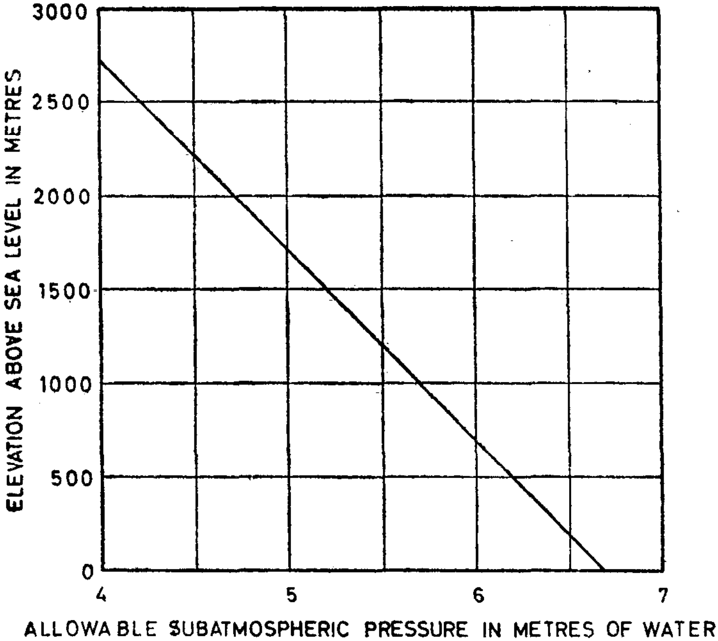

FIG. 1 ALLOWABLE SUBATMOSPHERIC PRESSURES FOR VARIOUS ELEVATIONS ABOVE SEA LEVEL

conditions prevailing in the tunnel, economics, construction limitations, etc. It is recommended that hydraulic model studies are conducted to determine an efficient and economical transition. The recommended shapes for entrance, contraction or expansion and exit transitions for pressure tunnels are given in **3.2** to **3.4**. However, for partly flowing tunnels the methods of design shall be the same as for open channel transition.

3.2 Entrance — To minimize head losses and to avoid zones where cavitation pressures may develop, the entrance to a pressure tunnel shall be streamlined to provide gradual and smooth changes in flow. To obtain best inlet efficiency the shape of entrance should simulate that of a jet discharging into air and should guide and support the jet with minimum interference until it is contracted to the tunnel dimensions. If the entrance curve is too sharp or too short, subatmospheric pressure areas which may induce cavitation, will develop. A bellmouth entrance which conforms to or slightly encroaches upon free jet profile will provide the best entrance shape.

3.2.1 For a circular tunnel the bellmouth shape may be approximated by an elliptical entrance curve represented by the following equation :

$$\frac{x^2}{(0{\cdot}5D)^2} + \frac{y^2}{(0{\cdot}15D)^2} = 1$$

where x and y are co-ordinates and D is the diameter of the tunnel at the end of entrance transition. The x-axis of the elliptical entrance is parallel to and at a distance of 0·65 D from the tunnel centre line; y-axis is normal to the tunnel centre line and 0·5 D downstream from the entrance face.

3.2.2 The jet issuing from a square or rectangular opening is not as easily defined as one issuing from a circular opening; the top and bottom curves may differ from the side curves both in length and curvature. Consequently, it is more difficult to determine a transition which will eliminate subatmospheric pressures. An elliptical curved entrance which will tend to minimize the negative pressure effects may be defined by the following equation :

$$\frac{x^2}{D^2} + \frac{y^2}{(0{\cdot}33\ D)^2} = 1$$

where D is the vertical height of the tunnel for defining the top and bottom curves, and also is the horizontal width of the tunnel for defining the side curves. The major and minor axes are positioned similar to those indicated for the circular bellmouth in **3.2.1**.

3.2.2.1 For a rectangular entrance with the bottom placed even with the upstream floor and with curved guide piers at each side of the entrance openings, both the bottom and side contractions will be suppressed and a sharper contraction will take place at the top of the opening. For this condition the top contraction curve may be defined by the following equation :

$$\frac{x^2}{D^2} + \frac{y^2}{(0{\cdot}67\ D)^2} = 1$$

where D is the vertical height of the tunnel downstream from the entrance.

3.3 Contraction and Expansion — To minimize head losses and to avoid cavitation tendencies along the tunnel surfaces, contraction and expansion transitions to and from gate control sections in a tunnel should be gradual. For contractions, the maximum convergent angle should not exceed that indicated by the relationship :

$$\tan \alpha = \frac{1}{U}$$

where

α = angle of the tunnel wall surfaces with respect to its centre line,

U = arbitrary parameter $\frac{v}{\sqrt{gD}}$,

v and D = average of the velocities and diameters at the beginning and end of the transition, and

g = acceleration due to gravity.

3.3.1 Expansion should be more gradual than contraction because of the danger of cavitation where sharp changes in the side walls occur. Furthermore, head loss coefficients for expansions increase rapidly after the angle α exceeds about 10°. Expansion should be based on the following relationship :

$$\tan \alpha = \frac{1}{2U}$$

The notations are the same as for equation given in **3.3**. For pressure tunnels, the angle α may not normally exceed 10°.

3.4 Exit — When a circular tunnel flowing partly full empties into a chute, the transition from the circular section to one with a flat bottom may be made in the open channel downstream from the tunnel portal, or it may be made within the tunnel so that the bottom will be flat at the portal section. Ordinarily, the transition should be made by gradually decreasing the circular quadrants from full radius at the upstream end of the transition to zero at the downstream end. For usual installations the length of the transition can be related to the exit velocity. An empirical rule which may be used to design a satisfactory transition for velocities up to 6 m/s is as follows:

$$L = \frac{2\,v\,D}{3}$$

where

L = length of transition in m,
v = exit velocity in m/s, and
D = tunnel diameter in m.

NOTE — For velocities higher than 6 m/s and depths greater than 5 m hydraulic model studies are essential.

4. PRESSURE FLOW LOSSES

4.1 Friction Losses — Friction factors for estimating the friction losses shall be based on actual field observations. For tunnels flowing full, friction loss may be computed by the use of the formula given in **4.1.1** and **4.1.2**.

4.1.1 *Manning's Formula* — The formula is given below :

$$v = \frac{R^{\frac{2}{3}}\,S^{\frac{1}{2}}}{n}$$

where

v = velocity in m/s,
R = hydraulic radius $\left(\dfrac{\text{area}}{\text{wetted perimeter}}\right)$ in m,
S = slope of energy gradient, and
n = roughness coefficient or rugosity coefficient.

4.1.1.1 For concrete lined tunnels the value of rugosity coefficient n varies from 0·012 to 0·018.

4.1.1.2 The value of rugosity coefficient n for use in the Manning's formula for an unlined tunnel depends on the nature of the rock and the quality of trimming, and is possibly influenced by the amount and distribution of overbreak. Recommended values of n for various rock surface conditions are given below:

Surface Characteristic	*Value of 'n'*	
	Min	*Max*
Very rough	0·04	0·06
Surface trimmed	0·025	0·035
Surface trimmed and invert concreted	0·020	0·030

NOTE — In a number of unlined tunnels the roughness has been experimentally determined by measuring discharges and friction losses or aerodynamically, data about which are given in Appendix A which may be used for design purpose assuming the effective area and overbreak.

4.1.2 *Darcy Waisbach Formula* — The formula is given below:

$$h_f = \frac{f\,L}{D} \times \frac{v^2}{2g}$$

where

h_f = friction headloss in m,
f = friction coefficient,
L = the length of the tunnel in m,
D = diameter of the tunnel in m,
v = velocity of flow in the tunnel in m/s, and
g = acceleration due to gravity in m/s².

NOTE — The formula given above is superior to the other empirical formulae, such as Bazin, Rehbock and Williams and Hazen because the friction factor f is dimensionless and no fractional powers are involved. The friction coefficient depends on the Raynolds number and the relative roughness, $\frac{K_s}{D}$ where K_s is the equivalent sand grain roughness.

4.1.2.1 For lined tunnels the value of f shall be computed in accordance with IS : 2951 (Part I)-1965*. The values of Ks, the equivalent sand grain roughness for concrete, may be adopted as below:

*Recommendations for estimation of flow of liquids in closed conduits: Part I Head loss in straight pipes due to frictional resistance.

Surface Characteristics	*Value of K_s* *mm*
Concrete Lining : Unusually rough Rough wood form work Erosion of poor concrete Poor alignment of joints	0·6 to 6·0
Rough Eroded by sharp materials in transit Marks visible from wooden forms Spalling of laitance	0·4 to 0·6
Granular Wood floated or brushed surface in good condition—good joints	0·18 to 0·4
New or fairly new—smooth concrete Steel forms—average workmanship Noticeable air voids on surface-smooth joints	0·06 to 0·18
New—unusually smooth concrete steel forms —first class workmanship Smooth joints	0·015 to 0·06

NOTE — The value of K_s for steel shall be taken from IS : 2951 (Part I)-1965*.

4.1.2.2 For unlined tunnels the value of f depends on the variation in cross-sectional area obtained in the field as well as the direction of driving the tunnel. Tests in, mostly, granite indicate that the friction loss may be estimated by measuring cross-sectional areas at intervals and determining the value of f by the following formula :

$$f = 0{\cdot}002\,57\ \delta$$

where

$$\delta = \frac{A_{99} - A_1}{A_1} \times 100$$

A_{99} = area corresponding to 99 percent frequency, and

A_1 = area corresponding to 1 percent frequency.

4.1.2.3 For tunnels of non-circular cross-section the diameter D in **4.1.2** shall be replaced by $4R$, where R is the hydraulic mean radius, thus reading as follows:

$$h_f = \frac{f L v^2}{8gR}$$

*Recommendations for estimation of flow of liquids in closed conduits: Part I Head loss in straight pipes due to frictional resistance.

4.1.3 For tunnels flowing partly full the head loss in friction shall be computed by the method specified in IS : 4745-1968*.

4.2 Trash Rack Losses — Trash rack structure which consists of widely spaced structural members without rack bars will cause very little head loss and trash rack losses in such a case may be neglected in computing tunnel losses. When the trash rack consists of a rack of bars, the loss will depend on bar thickness, depth and spacing and shall be obtained from the following formula :

$$h_t = K_t \frac{v^2}{2g}$$

where

h_t = trash rack head loss,

K_t = loss coefficient for trash rack

$= 1{\cdot}45 - 0{\cdot}45 \frac{a_n}{a_t} - \left[\frac{a_n}{a_t}\right]^2,$

a_n = net area through trash rack bars,

a_t = gross area of the vent (racks and supports),

v = velocity in net area, and

g = acceleration due to gravity.

4.2.1 Where maximum loss values are desired, 50 percent of the rack area shall be considered clogged. This will result in twice the velocity through the trash rack. For minimum trash rack losses, the openings may not be considered clogged when computing the loss coefficient or the loss may be neglected entirely.

4.3 Entrance Losses — Entrance loss shall be computed by the following equation :

$$h_e = K_e \frac{v^2}{2g}$$

where

h_e = head loss at entrance,

K_e = loss coefficient for entrance,

v = velocity, and

g = acceleration due to gravity.

4.3.1 Values of loss coefficient K_e for various types of entrances shall be assumed to be as given in Table 1.

4.4 Transition Losses — Head loss in gradual contractions or expansions in a tunnel may be considered in relation to the increase or decrease in velocity head and will vary according to the rate of change of area and

*Code of practice for design of cross-section of lined canals.

TABLE 1 LOSS COEFFICIENT FOR TUNNEL ENTRANCES

(*Clause* 4.3.1)

SL No.	TYPE OF ENTRANCE	LOSS COEFFICIENT FOR ENTRANCE, K_e		
		Maximum	Minimum	Average
(1)	(2)	(3)	(4)	(5)
i)	Gate in thin wall-unsuppressed contraction	1·80	1·00	1·50
ii)	Gate in thin wall-bottom and sides suppressed	1·20	0·50	1·00
iii)	Gate in thin wall-corners rounded	1·00	0·10	0·50
iv)	Square-cornered entrances	0·70	0·40	0·50
v)	Slightly rounded entrances	0·60	0·18	0·25
vi)	Fully rounded entrances $\frac{r}{D} \geqslant 0{\cdot}15$	0·27	0·08	0·10
vii)	Circular bellmouth entrances	0·10	0·04	0·05
viii)	Square bellmouth entrances	0·20	0·07	0·16
ix)	Inward projecting entrances	0·93	0·56	0·80

length of transition. These losses shall be assumed as specified in IS : 2951 (Part II)-1965*.

4.4.1 For gradual contractions, loss of head h_c, shall be computed by the following equation:

$$h_c = K_c \left[\frac{v_2^2}{2g} - \frac{v_1^2}{2g}\right]$$

where

K_c = loss coefficient for contraction,

v_2 = velocity in contracted section,

v_1 = velocity in normal section, and

g = acceleration due to gravity.

4.4.1.1 The value of loss coefficient K_c shall be assumed to vary from 0·1 for gradual contractions to 0·5 for abrupt contractions. Where flare angle does not exceed those specified in **3.3** the loss coefficient shall be assumed to be 0·1. For greater flare angles the loss coefficient shall be assumed to vary in straight line relationship to a maximum of 0·5 for a right angle contraction.

*Recommendations for estimation of flow of liquids in closed conduits: Part II Head loss in valves and fittings.

4.5 Bend and Junction Loss — Head loss at bends and junctions shall be assumed as given in IS : 2951 (Part II)-1965*.

4.6 Gate Loss in Pressure Tunnels — No gate loss need be assumed if the velocity of flow is less than 1 m/s. Where a gate is mounted at either the upstream or downstream side of a thin head wall such that the sides and bottom of jet are suppressed and the top is contracted, loss coefficients given in item (*ii*) of Table 1 shall be taken. Where a gate is so mounted in a tunnel that the floor, sides and the roof, both upstream and downstream, are continuous with the gate openings, only the losses due to the slot shall be considered as given below assuming the value of loss coefficient K_g not exceeding 0·10 :

$$h_g = K_g \frac{v^2}{2g}$$

where

h_g = gate head loss,
K_g = loss coefficient for gate,
v = velocity, and
g = acceleration due to gravity.

4.6.1 For partly open gates the coefficient of loss will depend on top contraction; for smaller openings it will approach a value of 1·0 as indicated in item (*ii*) of Table 1.

4.6.2 For wide open gates value of loss coefficient shall be assumed to be 0·19. Similar to partly open gates, value of the loss coefficient will increase for smaller gate openings.

4.7 Exit Losses — Where no recovery of velocity head will occur, such as where the release from a pressure tunnel discharges freely, or is submerged or supported on a downstream floor, velocity head loss coefficient K_{ex} shall be assumed to be equal to 1·0. Head loss at exit shall be computed by the following equation:

$$h_{ex} = K_{ex} \frac{v^2}{2g}$$

where

h_{ex} = exit head loss,
K_{ex} = loss coefficient for exit,
v = exit velocity, and
g = acceleration due to gravity.

4.7.1 Where a diverging tube is provided at the end of tunnel, recovery of a portion of the velocity head will be obtained if the tube expands gradually

*Recommendations for estimation of flow of liquids in closed conduits: Part II Head loss in valves and fittings.

and if the end of the tube is submerged, the loss coefficient K_{ex} shall be reduced from the value of 1·0 by the degree of head recovery.

5. VELOCITY

5.1 Average permissible velocity in a concrete lined tunnel may be about 6 m/s. For steel lined tunnels velocities as dictated by economic studies shall be chosen. In case of river diversion tunnels and tunnel spillways there may be no such limitations on the maximum permissible velocity, however, the lining and its surface shall be designed to withstand the velocities which will occur.

5.1.1 Permissible velocities in tunnels of different surfaces (unlined, concrete lined, steel lined) also depend upon the sediment load carried by the water. Where water carrying abrasive material in suspension and as bed load is to be conveyed the permissible velocity should be reduced. A recommended velocity is 2·5 m/s.

6. AIR LOCKING AND REMEDIAL MEASURE

6.1 General — The presence of air in a pressure tunnel can be a source of grave nuisance as given below:

a) The localization of an air pocket at the high point in a tunnel or at a change in slope which occasions a marked loss of head and diminution of discharge.
b) The slipping of a pocket of air in a tunnel and its rapid elimination by an air vent can provoke a water hammer by reason of the impact between two water columns.
c) The supply of emulsified water to a turbine affects its operation by a drop in output and efficiency thus adversely affecting the operation of generator. The presence of air in a Pelton nozzle can be the cause of water hammer shocks. Admission of air to a pump may occasion loss of priming.
d) If the velocity exceeds a certain limit air would be entrained causing bulking.

6.2 Source of Air — Air may enter and accumulate in a tunnel by the following means:

a) During filling, air may be trapped along the crown at high points or at changes in cross-sectional size or shape;
b) Air may be entrained at intake either by vortex action or by means of hydraulic jump associated with a partial gate opening; and
c) Air dissolved in the flowing water may come out of solution as a result of decrease in pressure along the tunnel.

6.3 Remedial Measures — The following steps are recommended to prevent the entry of air in a tunnel:

a) Shallow intakes are likely to induce air being sucked in. Through-

out the tunnel the velocity should either remain constant or increase towards the outlet end. It should be checked that at no point on the tunnel section negative pressures are developed.

b) Vortices that threaten to supply air to a tunnel should be avoided, however, if inevitable they should be suppressed by floating baffles, hoods or similar devices.

c) Partial gate openings that result in hydraulic jumps should be avoided.

d) Traps or pockets along the crown should be avoided.

6.3.1 In some cases, such as secondary feeder shafts supplying a main tunnel air entrance may appear inevitable. In such cases de-aeration chamber with enlarged area should be provided so that no air enters the main tunnel. Where possible it is advisable that such intakes are checked on hydraulic models to ensure no entrance of air.

APPENDIX A

(*Note under Clause* 4.1.1.2)

VALUES OF *n* FOR EXISTING TUNNELS

Sl No.	Type of Rock	Theoretical		Actual Effective		Overbreak		Manning's n
		Area (A_t)	Hydraulic Radius (R)	Area (A_e)	Hydraulic Radius (R_e)	$\frac{A_e}{A_t}$	Percent (Volume)	
		m²	m	m²	m			
i)	Granite-gneiss	30	1·46	33·8	1·54	1·128	12·7	0·035 4*
ii)	Granite-gneiss	50	1·85	57·4	2·09	1·15	14·8	0·034 3*
iii)	Granite-gneiss	50	1·85	61·5	2·16	1·23	23·0	0·030 0*
iv)	Granite-gneiss	30	1·45	35·9	1·62	1·20	19·6	0·038 4*
v)	Gneiss-granite with some diabase	60	2·00	64·0	—	1·07	6·6	0·027 0*
vi)	Vein-gneiss	5	0·59	6·6	0·71	1·32	32·0	0·033 9*
vii)	Arkose sand-stone	28·7	1·53	34·9	1·64	1·21	—	0·038 †
viii)	Arkose sand-stone	35·4	1·68	40·1	1·74	1·13	—	0·038 †
ix)	Upper sillurian slate horizontally stratified	105	2·74	114·3	2·88	1·088	8·9	0·029 2*
x)	Black slate with granite intrusions	70	2·24	80·5	2·42	1·15	15·1	0·043 7*

*Calculated from the length of tunnel, the effective area and the hydraulic radius and he observed friction head.

†Calculated from the length of tunnel, from actual area of tunnel and hydraulic radius of quivalent circle.

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CODE OF PRACTICE FOR DESIGN IN TUNNELS CONVEYING WATER

PART IV STRUCTURAL DESIGN OF CONCRETE LINING IN ROCK

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UDC 624.196 : 624.191.1

BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
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Indian Standard

CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART IV STRUCTURAL DESIGN OF CONCRETE LINING IN ROCK

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AMENDMENT NO. 1 JUNE 1986

TO

IS:4880(Part 4)-1971 CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART 4 STRUCTURAL DESIGN OF CONCRETE LINING IN ROCK

(*Page 24, clause D-2.1, line 7*) - Substitute the following for the existing line:

'm_1, m_2 = Poisson's number of rock and concrete respectively.'

(*Page 24, clause D-2.1, lines 13 and 14*) - Substitute the following in the existing lines:

'a = internal radius of the tunnel; and

b = external radius of the lining up to A-line.'

(BDC 58)

Printed at New India Printing Press, Khurja, India

AMENDMENT NO. 2 JANUARY 2008
TO
IS 4880 (PART 4) : 1971 CODE OF PRACTICE FOR .DESIGN IN TUNNELS CONVEYING WATER

PART 4 STRUCTURAL DESIGN OF CONCRETE LINING IN ROCK

(*Page* 5, *clause* 3.1) — Substitute 'IS 456 : 2000*' *for* 'IS 456 : 1964*'.

(*Page* 9, *clause* 6.1) — Substitute 'IS 456 : 2000*' *for* 'IS 456 : 1964*'.

(*Pages* 5 *and* 9, *footnote marked* *) — Substitute the following for the existing footnote:

'Plain and reinforced concrete — Code of practice (*fourth revision*) '

(WRD 14)

Reprography Unit, BIS, New Delhi, India

Indian Standard

CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART IV STRUCTURAL DESIGN OF CONCRETE LINING IN ROCK

0. FOREWORD

0.1 This Indian Standard (Part IV) was adopted by the Indian Standards Institution on 15 March 1971, after the draft finalized by the Water Conductor Systems Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 Water conductor system occasionally takes the form of tunnels through high ground or mountains, in rugged terrain where the cost of surface pipe line or canal is excessive and elsewhere as convenience and economy dictates. This standard, which is being published in parts, is intended to help engineers in design of tunnels conveying water. This part lays down the criteria for structural design of concrete lining for tunnels in rock, covering recommended methods of design. However, in view of the complex nature of the subject, it is not possible to cover each and every possible situation in the standard and many times a departure from the practices recommended in this standard may be considered necessary to meet the requirements of a project or site for which descretion of the designer would be required.

0.3 This standard is one of a series of Indian Standards on tunnels. (*see* page 28).

0.4 Other parts of this standard are as follows:

- Part I General design
- Part II Geometric design
- Part III Hydraulic design
- Part V Structural design of concrete lining in soft strata and soils
- Part VI Tunnel supports

0.5 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS : 2-1960*. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

*Rules for rounding off numerical values (*revised*).

1. SCOPE

1.1 This standard (Part IV) covers criteria for structural design of plain and reinforced concrete lining for tunnels and circular shafts in rock mainly for river valley projects.

NOTE — The provisions may, nevertheless, be used for design of any other type of tunnel, like railway or roadway tunnel, provided that all the factors peculiar to such projects which may affect the design are taken into account.

1.2 This standard, however, does not cover the design of steel and prestressed concrete linings, tunnels in swelling and/or squeezing type of rocks, soils or clays and cut and cover sections.

2. TERMINOLOGY

2.0 For the purpose of this standard, the following definitions shall apply.

2.1 Minimum Excavation Line (A-Line) — A line within which no unexcavated material of any kind and no supports other than permanent structural steel supports shall be permitted to remain (*see* Fig. 1).

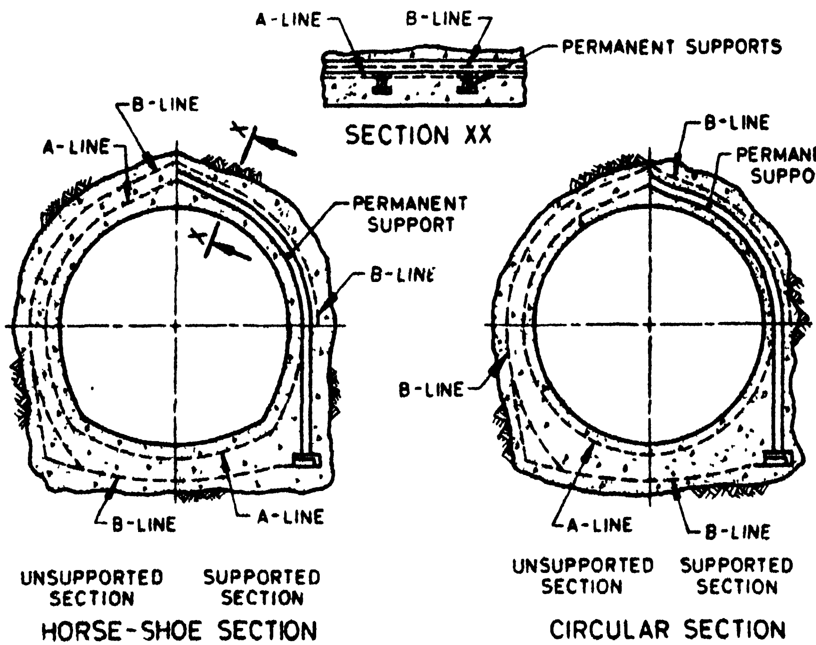

FIG. 1 TYPICAL SECTIONS OF CONCRETE LINED TUNNELS SHOWING A- AND B-LINES

2.2 Pay Line (B-Line) — An assumed line (beyond A-line) denoting mean line to which payment of excavation and concrete lining is made whether the actual excavation falls inside or outside it.

2.3 Cover — Cover on a tunnel in any direction is the distance from the tunnel soffit to the rock surface in that direction. However, where the thickness of the overburden is sizable its equivalent weight may also be reckoned provided that the rock cover is more than three times the diameter of the tunnel.

3. MATERIAL

3.1 Use of plain and reinforced concrete shall generally conform to IS:456-1964*.

4. GENERAL

4.1 Structural design of tunnel lining requires a thorough study of the geology of rock mass, the effective cover, results of *in situ* tests for modulus of elasticity, poissons ratio, state of stress and other mechanical characteristics of the rock. It is preferable to make a critical study of these factors which may be done in pilot tunnels, test drifts, during actual excavations or by other exploratory techniques. The assessment of rock load on the lining and portion of the internal pressure which should be assumed to be transmitted to the rock mass, will have to be done by the designer on the basis of the results of these investigations.

4.2 It is essential for the designer to have fairly accurate idea of the seepage, and the presence or absence of ground water under pressure likely to be met with. Where heavy seepage of water is anticipated, the designer shall make provisions for grouting with cement and/or chemicals or extra drainage holes, and also consider the feasibility of providing steel lining, if necessary. It is recommended that such designs of alternate use of steel lining be made along with the design of plain or reinforce lining so that a design is readily available should the construction personnel require it when they meet unanticipated conditions.

4.3 The portions of a tunnel which should be reinforced and the amount of reinforcement required depends on the physical features of the tunnel, geological factors and internal water pressure. For a free-flow tunnel normally no reinforcement need be provided. However, reinforcement shall be provided where required to resist external loads due to unstable ground or grout or water pressures. Pressure tunnels with high hydrostatic loads shall have lining reinforced sufficiently to withstand bursting where inadequate cover or unstable supporting rock exists.

*Code of practice for plain and reinforced concrete (*second revision*).

4.3.1 The provision of reinforcement in the tunnel lining complicates the construction sequence besides requiring a thicker lining. The use of reinforcement should, therefore, be restricted, as much as possible consistent with the safety of lining. For free flow tunnels it is recommended that as far as possible, no reinforcement should be provided to resist external loads. Such loads should be resisted and taken care of by steel supports and/or precast concrete rings.

4.3.2 A pressure tunnel should ordinarily be reinforced wherever the depth of cover is less than the internal pressure head. The final choice whether reinforcement should be provided or not would be guided by the geological set up and economics.

4.3.3 For design of junctions and transitions for tunnels detailed structural analysis shall be made. Such transitions are difficult to construct in the restricted working space in tunnels, and the designer shall keep in view this aspect and propose structures which are easy for construction.

4.3.4 An adequate amount of both longitudinal and circumferential reinforcement in addition to steel supports may be provided, if required, near the portals of both pressure and free-flow tunnels to resist loads resulting from loosened rock headings or from sloughing of the portal cuts.

4.4 If the seepage of water through the lining is likely to involve heavy loss of water and the structural stability of the rock mass around the tunnel is likely to be affected adversely or might lead to such situation as to be damaging to the tunnel or adjoining structures, steel lining shall be provided. Where rock cover is less than that specified in **4.5** and **4.5.1** or where the cavitation of lining is expected due to the high velocity of water or erosion is expected the provision of a steel liner shall be considered.

4.5 Where the rock is relatively impervious and the danger of blow out exists, the vertical cover shall be greater than the internal pressure head in the tunnel. In other cases the weight of the rock over the tunnel shall be greater than the internal pressure.

NOTE — The conventional practice is to provide a vertical cover equal to the internal water pressure head (H). Recent trend, however, is to provide lesser cover (as low as 0·5 H) depending upon the nature of the rock.

4.5.1 For tunnels located near mountain slopes, the lateral cover rather than the vertical cover may be the governing criteria many times. In such cases the effective vertical cover equivalent to the actual lateral cover shall be found out, by drawing a profile of the ground surface (perpendicular to the contour lines) and fitting the curve shown in Fig. 2 in such a way that it touches the ground surface. The vertical distance

marked 'C_v' shall be designated as the effective vertical cover and shall be greater than the internal pressure head in the tunnel. This method of estimating the effective cover shall not hold good where joints, stratification, faults, etc, in the rock are adversely located to invalidate the assumption that horizontal cover is half as effective as the vertical cover on the basis of which the curve of Fig. 2 is drawn. In such cases special analysis shall be made for determining the stability of the rock mass around the tunnel.

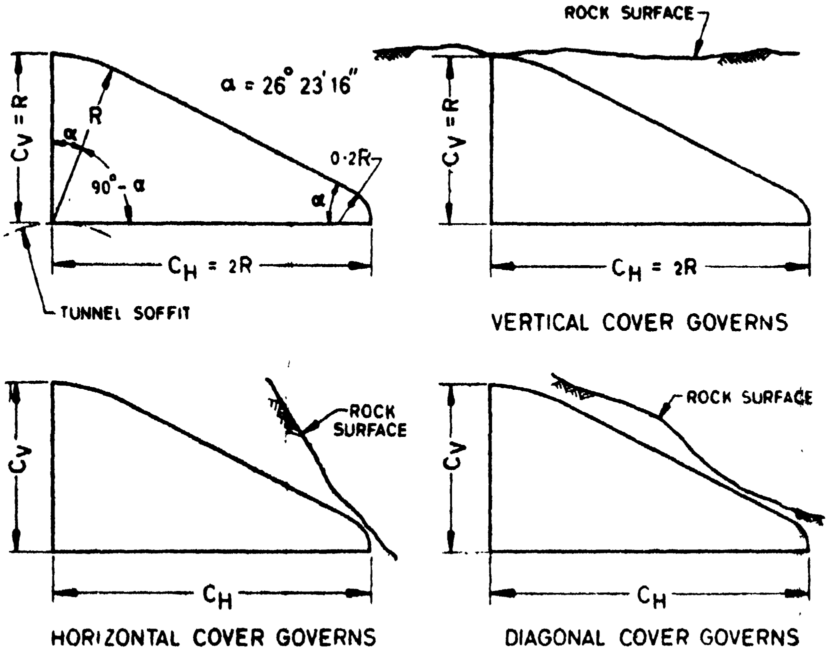

FIG. 2 ESTIMATION OF EFFECTIVE COVER

5. LOADING CONDITIONS

5.1 General — Design shall be based on the most adverse combination of probable load conditions, but shall include only those loads which have reasonable probability of simultaneous occurrence.

5.2 Load Conditions — The design loading applicable to tunnel linings shall be classified as 'Normal' and 'Extreme' design loading conditions. Design shall be made for 'Normal' loading conditions and shall be checked

for safety under 'Extreme' loading conditions (*see* Appendix A). The design loading shall be as follows:

a) *External Rock Load* (*see* **7.4.2**)

b) *Self Load of Lining*

c) *Design External Water Pressure* (*see* **7.4.3**):

   1) *Normal design loading conditions* — The maximum loading obtained from either maximum steady or steady state condition with loading equal to normal maximum ground water pressure and no internal pressure, or maximum difference in levels between hydraulic gradient in the tunnel, under steady state or static conditions and the maximum down surge under normal transient operation.

   2) *Extreme design loading conditions* — Loading equal to the maximum difference in levels between the hydraulic gradient in the tunnel under static conditions and the maximum down surge under extreme transient operations or the difference between the hydraulic gradient and the tunnel invert level in case of tunnel empty condition.

d) *Design Internal Water Pressure*:

   1) *Normal design loading conditions* — Maximum static conditions corresponding to maximum water level in the head pond, or loading equal to the difference in levels between the maximum upsurge occurring under normal transient operation and the tunnel invert.

   2) *Extreme design loading conditions* — Loading equal to the difference between the highest level of hydraulic gradient in the tunnel under emergency transient operation and the invert of the tunnel.

e) *Grout Pressures* (*see* **7.4.4**)

f) *Other Loads* — Pressure transmitted from buildings and structures on external surface lying within the area of subsidence and non-permanent loads, such as weight of vehicles moving in the tunnel or on the surface above it, where applicable.

5.3 The loading conditions vary from construction stage to operation stage and from operation stage to maintenance stage. The design shall be checked for all probable combination of loading conditions likely to occur during all these stages.

6. STRESSES

6.1 For design of concrete lining, the thickness of concrete up to A-line shall be considered. The stresses for concrete and reinforcement shall be in accordance with IS:456-1964* for design of lining for condition of normal load.

6.1.1 For extreme conditions of loading, the stresses in accordance with **6.1** shall be increased by 33⅓ percent.

7. DESIGN

7.1 The design of concrete lining for external loads may be done by considering it as an independent structural member (*see* **7.4**). The design of concrete lining for internal water pressure may be done by considering the lining as a part of composite thick cylinder consisting of peripheral concrete and surrounding rock mass subjected to specific boundary conditions (*see* **7.5**).

NOTE — To ensure the validity of the assumption that lining is a part of composite thick cylinder in the latter case adequate measures shall be taken, such as pressure grouting of the rock mass surrounding the tunnel.

7.2 Thickness of Lining — The thickness of the lining shall be designed such that the stresses in it are within permissible limits when the most adverse load conditions occur. The minimum thickness of the lining will, however, be governed by requirements of construction. It is recommended that the minimum thickness of unreinforced concrete lining be 15 cm for manual placement. Where mechanical placement is contemplated the thickness of the lining shall be so designed that the slick line can be easily introduced on the top of the shutter without being obstructed by steel supports. For a 15-mm slick line a clear space of 18 cm is recommended. For reinforced concrete lining, a minimum thickness of 30 cm is recommended, the reinforcement, however, being arranged in the crown to allow for proper placement of slick line.

7.2.1 However, for preliminary design of lining for tunnels in reasonably stable rock, a thickness of lining may be assumed to be 6 cm per metre of finished diameter of tunnel.

NOTE — Minimum thickness of lining, as necessary from structural considerations, should be provided since thin linings are more flexible and shed off loads to the abutments.

7.3 Where structural steel supports are used, they shall be considered as reinforcement only if it is possible to make them effective as reinforcement by use of high tensile bolts at the joints or by welding the joints. A

*Code of practice for plain and reinforced concrete (*second revision*).

minimum cover of 15 cm shall be provided over the inner flange of steel supports and a minimum cover of 8 cm over the reinforcement bars.

NOTE — Welding of joints in soft strata tunnels may not be possible, and it may become necessary to embed the steel supports partly or fully in primary concrete immediately after erection. Welded joints should, therefore, be avoided.

7.4 Design for External Loads — The lining may be considered as an independent structural element assuming that it deflects under the active external loads and its deflection is restricted by the passive resistance developed in the surrounding rock mass. At any point on the perifery of the lining this may be stated by the following equation:

$$\Delta p = \Delta r + \Delta e + \Delta w - \Delta y$$

where

Δp = deflection due to passive resistance;

Δr = deflection due to rock load;

Δe = deflection due to self weight of lining and the water contained in it;

Δw = deflection due to the external water pressure, if any; and

Δy = yield of surrounding rock mass due to abutting of the lining against it.

7.4.1 The following loads and reactions are involved in the design of lining:

a) Rock load (*see* **7.4.2**);

b) External pressure of water, if any (*see* **7.4.3**);

c) Grout pressure, if any (*see* **7.4.4**);

d) Self weight of lining;

e) Weight of water contained in the tunnel;

f) Reaction due to active vertical loads (*see* **7.4.5**); and

g) Lateral passive pressure due to the deformation of lining (*see* **7.4.6**).

NOTE — As compared to the other loads, the self weight and the weight of water contained in the tunnel are small. These loads are not discussed in detail. However, the formulae for calculating bending moment, thrust, radial shear and horizontal and vertical deflections caused by self load and the weight of water contained in the conduit are given in Appendix C.

7.4.2 *Rock Load* — Rock load acting on the tunnel varies depending upon the type and mechanical characteristics of rock mass pre-existing stresses in the rock mass and the width of the excavation. The rock load is also affected by ground water conditions which may lubricate the joints in rock and cause greater load than when the material is dry.

The existing stresses and their distribution after tunnelling has a great effect on the development of load on tunnel lining. The redistribution may take considerable time to reach an equilibrium condition. Wherever it is felt that rock loads are likely to develop excessively it is advisable to prevent the movement of rock by immediately supporting it by shotcreting and/or steel supports and primary lining. Where weak rocks are supported by steel rib supports, much of the rock load would be taken by the supports and the lining would take the load developed after its placement. There may also be redistribution of stresses in supports due to deformation of the lining. A reasonable approach is to determine the diametral changes in the supported section with respect to time before concreting to estimate the extent of deformation that has already taken place. This investigation may be conveniently done in an experimental section of the tunnel or pilot tunnel with proper instrumentation.

In major tunnels, it is recommended that as excavation proceeds load cell measurements and diametral change measurements are carried out to estimate the rock loads. In rocks where the loads and deformations do not attain stable values, it is recommended that pressure measurements should be made using flat jack or pressure cells.

7.4.2.1 In the absence of any data and investigations, rock loads may be assumed to be acting uniformly over the tunnel crown as shown in Fig. 3 in accordance with Appendix B. However, Appendix B may be taken as an aid to judgement by the designer.

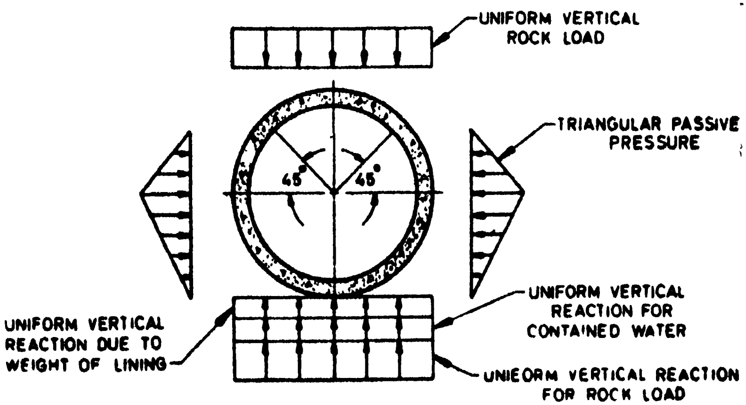

FIG. 3 EXTERNAL LOADS ON LINING

7.4.3 *External Water Pressure* — Lining shall be designed for external water pressure, if any. However, in areas where drainage holes are provided, lining shall not be designed for external water pressure (*see* **8.1**). For

conditions of loading for external water pressure reference may be made to **5.2**. For design of lining for external water pressure where effective pressure grouting has been done, the water pressure may be assumed to act on the whole grouted cylinder which may be taken as a structural element for computing the stresses and deformation of lining.

7.4.4 *Grout Pressures* — The lining shall be checked for stresses developed in it at the pressure on which grouting is done and it shall be ensured that the stresses are within the limits depending on the strength attained by concrete by then (*see* **9.1** and **9.2**).

7.4.5 *Vertical Reactions* — Reactions due to the vertical loads may be assumed, reasonably, to be vertical and uniformly distributed on the invert of the lining as shown in Fig. 3.

NOTE — In fact, these reactions, may, however, not be uniform depending upon the foundation conditions. Nevertheless, with the uncertainties involved in the design of tunnels, the assumption may not give far out results. Moreover, normally, uniform reactions would give more critical condition.

7.4.6 *Lateral Passive Pressure* — The lateral passive pressure may be estimated either by considering the lining as a ring restrained by elastic medium with a suitable modulus of deformation or by restricting the maximum deflection of the lining at the horizontal diameter to an assumed value (depending upon the yield of the surrounding rock mass) by the lateral passive pressure. The latter method is discussed in detail in **7.4.6.1** for design of circular linings. For design of non-circular linings the former method shall only be used.

7.4.6.1 For analyzing a circular lining, the designer will have to assume the maximum deflection to be permitted along the horizontal central axis of the tunnel. This value of horizontal deflection will consist of the yield of the rock mass surrounding the tunnel (*see* Note 2). It may be assumed that any further deflection of the lining is restricted to this value by the lateral passive resistance of the surrounding rock mass in a pattern given in Fig. 3. The design shall be such that the maximum value of the passive resistance is within the maximum permissible values.

NOTE 1 — For a circular lining the deflection is outward in a zone extending approximately from 45° above the horizontal diameter to the invert as the invert move up. The maximum value is near horizontal diameter. On the above consideration and neglecting the effect of vertical translation of the lining, the lateral rock restraint may be assumed to have approximately a straingular distribution as shown in Fig. 3. The vertical translation of the lining would cause a shifting of the point of maximum intensity slightly below the horizontal diameter and restriction of the upper limit to an angle slightly less than 45°, but the effect of vertical translation would be small and can be neglected. The passive pressures would, in fact, be also radial to the surface but in view of the fact that the deflections would be mostly in horizontal direction and vertical deflections would be small, it may be assumed to act in horizontal direction.

NOTE 2 — The rock mass surrounding the tunnel yields due to the bearing pressure of lining against it. This yield may be due to closing up of joints and fractures in the rock mass and also due to its elastic or plastic deformation. The value of yield taken for design should be based on the experiments carried out in the test sections (*see* **4.1**).

At Bhakra Dam, rock deflection tests indicated yield of either face in poor shattered rock as 1·25 mm under a stress of 54 kg/cm². However, a total deflection of 3·8 mm was assumed in the design of lining though actual bearing pressure on the rock was anticipated to be much less. Experiments on Garrison Dam tunnels, in clay shale of internal diameter 8·8 m and lining thickness of 0·9 m indicated deflection of either face of lining at horizontal diameter to range between 3 to 4 mm. In design of Ramganga Dam tunnels outer deflection of either face of lining at horizontal diameter was assumed to be 3·8 mm.

7.4.6.2 Formulae for determining the values of horizontal deflection, vertical deflection, bending moments, normal thrust, radial shear for the various circumferential points on a circular lining are given in Appendix C considering the invert as the reference point for various loads excepting the internal pressure. In derivation of these values the pattern of vertical reaction and passive pressure has been assumed in accordance with **7.4.5** and **7.4.6** as shown in Fig. 3.

7.4.7 For non-circular lining model tests are recommended to determine the stress distributions. However, the design may be done assuming uniformly distributed loads as in the case of circular tunnels, and using the same distribution for passive pressures. The design of such indeterminate sections may be done by standard methods and is not covered by this standard.

7.5 Design For Internal Water Pressure — Lining shall be considered as a part of composite thick cylinder consisting of peripheral concrete and surrounding rock mass subjected to specified boundary conditions.

NOTE — This method suffers from uncertainties of external loads, material properties and indeterminate tectonic forces. In this method the rock surrounding the tunnels is assumed to have reasonably uniform characteristics and strength and that effective pressure grouting has been done to validate the assumption that concrete lining and surrounding rock behave as a composite cylinder. The grout fills the cracks in the rock and thus reduces its ability to deform inelastically and increases the modulus of deformation. If the grout pressures are high enough to cause sufficient prestress in the lining the effect of temperature and drying shrinkage and inelastic deformation might be completely counteracted.

7.5.1 For analysing a circular lining the method given in Appendix D may be adopted. The design shall be such that at no point in the lining and the surrounding rock the stresses exceed the permissible limits.

If the rock is very good, and cracking of lining is not otherwise harmful, cracking of the lining may be permitted to some extent. In that case, tangential stress in concrete lining will be absent and correspondingly, tangential stress in rock will increase.

If the rock is not good, tensile stress in concrete may exceed the allowable limit and in such a case, reinforcement may be provided. Reinforcement however, is not capable of reducing the tensile stresses to a considerable extent. By suitable arrangement, it will help to distribute the cracks on the whole periphery in the form of hair cracks which are not harmful because they may get closed in course of time, or at least they will not result in serious leakages.

7.5.2 For analyzing non-circular linings, the stress pattern may be determined by photo-elastic studies.

8. GROUND WATER DRAINAGE HOLES

8.1 Drainage holes may be provided in other than water conveying tunnels to relieve external pressure, if any, caused by seepage along the outside of the tunnel lining. In free flow tunnels drainage holes may be provided in the crown above the full supply level. In case of pressure tunnels, if the external water pressure is substantially more than the internal water pressure, drainage holes may be provided in the crown. However, when the mountain material is likely to be washed into the tunnel through such drainage holes they shall not be provided.

8.1.1 The arrangement of drainage holes depends upon the site conditions and shall be decided by the designer. A recommended arrangement is described below:

> At successive sections; one vertical hole drilled in the crown alternating with two drilled horizontal holes one in each side wall extending to a depth of at least 15 cm beyond the back of the lining.

8.1.2 If the flow through the tunnel is conveyed in a separate pipe, the horizontal holes shall be drilled near the invert.

9. GROUTING

9.1 Backfill Grouting — Backfill grouting shall be done at a pressure not exceeding 5 kg/cm^2 and shall be considered as a part of concreting. It shall be done throughout the length of the concrete lining not earlier than 21 days after placement after the concrete in the lining has cooled off. However, stresses developed in concrete at the specified grout pressure may be calculated and seen whether they are within permissible limits depending on the strength attained by concrete by then. The grout pressure mentioned above is the pressure as measured at the grout hole.

NOTE — Backfill grouting serves to fill all voids and cavities between concrete lining and rock.

9.2 Pressure Grouting — Pressure grouting shall be done at a maximum practicable pressure consistent with the strength of lining and safety against uplift of overburden. The depth of grout holes shall be at least equal to the diameter of the tunnel.

NOTE 1 — Pressure grouting consolidates the surrounding rock and fills any gaps caused by shrinkages of concrete. This grouting is normally specified where lining is

reinforced, to improve the rock quality and, therefore, to increase the resistance of rock to carry internal water pressure. As a rule of thumb a grout pressure of 1·5 times the water pressure in the tunnel may be used subject to the conditions that safety against uplift of the overburden is ensured. Grout pressures of up to 5 to 10 times the water pressure in the tunnel have been used in Italy.

NOTE 2 — It is advantageous to provide a grout curtain by means of extensive deep grouting at the reservoir end of the tunnel to reduce loss of water due to seepage.

9.3 Pattern of Holes for Grouting — For small tunnels, rings of grout holes, may be spaced at about 3 m centres, depending upon the nature of the rock. Each ring may consist of four grout holes distributed at about 90° around the periphery, with alternate rings placed vertical and 45° axes.

APPENDIX A

(*Clause* 5.2)

BASIC CONDITIONS FOR INCLUDING THE EFFECT OF WATER HAMMER IN THE DESIGN

A-1. The basic conditions for including effect of water hammer in the design of tunnels or turbine penstock installations are divided into normal and emergency conditions with suitable factors of safety assigned to each type of operation.

A-2. NORMAL CONDITIONS OF OPERATIONS

A-2.1 The basic conditions to be considered are as follows:

a) Turbine penstock installation may be operated at any head between the maximum and the minimum values of forebay water surface elevation.

b) Turbine gates may be moved at any rate of speed by action of the governor head up to a predetermined rate, or at a slower rate by manual control through the auxiliary relay valve.

c) The turbine may be operating at any gate position and be required to add or drop any or all of its load.

d) If the turbine penstock installation is equipped with any of the following pressure control devices it will be assumed that these devices are properly adjusted and function in all manner for which the equipment is designed.

   1) Surge tanks,

   2) Relief valves,

   3) Governor control apparatus,

   4) Cushioning stroke device, and

   5) Any other pressure control device.

e) Unless the actual turbine characteristics are known, the effective area through the turbine gates during the maximum rate of gate movement will be taken as a linear relation with reference to time.

f) The water hammer effects shall be computed on the basis of governor head action for the governor rate which is actually set on the turbine for speed regulation. If the relay valve stops are adjusted to give a slower governor setting than that for which the governor is designed this shall be determined prior to proceeding with the design of turbine penstock installation and later adhered to at the power plant so that an economical basis for designing the penstock scroll case, etc, under normal operating conditions may be established.

g) In those instances, where due to higher reservoir elevation, it is necessary to set the stops on the main relay valve for a slower rate of gate movement, water hammer effects will be computed for this slower rate of gate movement also.

h) The reduction in head at various points along the penstock will be computed for rate of gate opening which is actually set in the governor in those cases where it appears that the profile of the penstock is unfavourable. This minimum pressure will then be used as a basis for normal design of the penstock to ensure that sub-atmospheric pressures will not cause a penstock failure due to collapse.

j) If a surge is present in the penstock system, the upsurge in the surge tank will be computed for the maximum reservoir level condition for the rejection of the turbine flow which corresponds to the rated output of the generator during the gate traversing time which is actually set on the governor.

k) The downsurge in the surge tank will be computed for minimum reservoir level condition for a load addition from speed-no-load to the full gate position during the gate traversing time which is actually set on the governor.

A-3. EMERGENCY CONDITIONS

A-3.1 The basic conditions to be considered as an emergency operation are as follows:

a) The turbine gates may be closed at any time by the action of the governor head, manual control knob with the main relay valve or the emergency solenoid device.

b) The cushioning stroke will be assumed to be inoperative.

c) If a relief valve is present, it will be assumed inoperative.

d) The gate traversing time will be taken as the minimum time for which the governor is designed.

e) The maximum head including water hammer at the turbine and along the length of the penstock will be computed for the maximum reservoir head condition for final part gate closure to the zero gate position at the maximum governor rate in $\frac{2L}{a}$ seconds. Where 'L' is length of penstock and 'a' wave velocity.

f) If a surge tank is present in the penstock system, the upsurge in the tank will be computed for the maximum reservoir head condition for the rejections of full gate turbine flow at the maximum rate for which the governor is designed. The downsurge in the surge tank will be computed for the minimum reservoir head condition for full gate opening from the speed-no-load position at the maximum rate for which the governor is designed. In determining the top and bottom elevations of the surge tank nothing will be added to the upsurge and down surge for this emergency condition of operation.

A-4. EMERGENCY CONDITIONS NOT TO BE CONSIDERED AS A BASIS FOR DESIGN

A-4.1 The other possible emergency conditions of operation are those during which certain pieces of control are assumed to malfunction in the most unfavourable manner. The most severe emergency head rise in a turbine penstock installation occurs from either of the two following conditions of operation:

a) Rapid closure of turbine gates in less than $\frac{2L}{a}$ s, when the flow of water in the penstock is maximum.

b) Rhythmic opening and closing of the turbine gates when a complete cycle of gate operation is performed in $\frac{4L}{a}$ s.

Since these conditions of operation require a complete malfunctioning, of the governor control apparatus at the most unfavourable moment, the probability of obtaining this type of operation is exceedingly remote. Hence, the conditions shall not be used as a basis for design. However, after the design has been established from other considerations it is desirable that the stresses in the turbine scroll case penstock and pressure control devices be not in excess of the ultimate bursting strength or twisting strength of structures for these emergency conditions of operation.

APPENDIX B

(*Clause* 7.4.2.1)

ROCK LOADS ON TUNNEL LINING

B-1. SCOPE

B-1.1 This appendix contains recommendations for evaluating rock loads on tunnel lining.

B-2. LOAD DISTRIBUTION

B-2.1 Rock load may be assumed as an equivalent uniformly distributed load over the tunnel soffit over a span equal to the tunnel width or diameter as the case may be.

B-3. LOAD

B-3.1 Rock loads may be estimated by any of the methods given in **B-3.1.1** to **B-3.1.3**.

B-3.1.1 Rock load at depths less than or equal to 1·5 ($B + H_t$) may be taken as equal to depth of actual rock cover where B is the width and H_t is the height of the tunnel opening. In case of circular tunnels B and H_t both will be equal to the diameter of tunnel D.

Rock load (H_p) on the roof of support in tunnel with width B and height H_t at depth of more than 1·5 ($B+H_t$) may be assumed to be according to Table 1.

B-3.1.2 Rock load may also be worked out using Fenner's ellipse (*see* Fig. 4 on P 21) by the following equation:

$$a = \frac{b}{2}(m - 2)$$

where

a = main axis of ellipse of rock load,

b = tunnel diameter (excavated), and

m = inverse of poissons ratio for rock (which usually varies from 2 to 7).

The weight of rock in the shaded portion may be taken as rock load and may be considered as uniformly distributed on the diameter of tunnel.

NOTE — The above treatment assumes the rock to be homogeneous and to behave within elastic range. It has also limited application as it does not give any rock loads for value of poissons ratio equal to 0·25 and above. It does not also take into consideration the strength and characteristics of rock.

B-3.1.3 The rock load according to the Russian practice depends upon the degree of rock firmness. The rock load may be taken as that for rock area enclosed by a parabola starting from intersection points of the rupture planes with horizontal length drawn to the crown of the tunnel section. The dimensions of the parabola are given below (*see* Fig. 5 on P 21):

$$h = \frac{B}{2f}$$

$$B = b + 2\,m \tan\,(45° - \phi/2)$$

where

ϕ = the angle of repose of the soil,

f = the strength factor after Protodyakonov (*see* Table 2).

In the case of circular tunnels,

$$B = D\,[\,1 + 2 \tan\,(45° - \phi/2)\,]$$

where

D = diameter of the tunnel.

TABLE 1 ROCK LOAD

(*Clause* B-3.1.1)

Rock Condition	Rock Load H_p m	Remarks
1. Hard and intact	Zero	Light lining required only if spalling or popping occurs
2. Hard stratified or schistose	0 to 0·50 B	Light support
3. Massive, moderately jointed	0 to 0·25 B	Load may change erratically from point to point
4. Moderately blocky and seamy	0·25 B to 0·35 ($B + H_t$)	No side pressure
5. Very blocky and seamy	(0·35 to 1·10) ($B + H_t$)	Little or no side pressure
6. Completely crushed but chemically intact	1·10 ($B + H_t$)	Considerable side pressure. Softening effect of seepage towards bottom of tunnel. Requires either continuous support for lower ends of ribs or circular ribs.
7. Squeezing rock, moderate depth	(1·10 to 2·10) ($B+H_t$)	Heavy side pressure. Invert struts required. Circular ribs are recommended.
8. Squeezing rock, great depth	(2·10 to 4·50) ($B+H_t$)	
9. Swelling rock	Up to 80 m irrespective of value of ($B + H_t$)	Circular ribs required. In extreme cases use yielding support

NOTE 1 — The above Table has been arrived on the basis of observation and behaviour of supports in Alpine tunnels where the load was derived mainly on loosening type of rock.

NOTE 2 — The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4 to 6 may be reduced by fifty percent.

NOTE 3 — Some of the most common rock formations contain layers of shale. In an unweathered state, real shales are no worse than other stratified rocks. However, the term shale is often applied to firmly compacted clay sediments which have not yet acquired the properties of rock. Such so called shales may behave in the tunnel like squeezing or even swelling rock.

NOTE 4 — If rock formation consists of sequence of horizontal layers of sand stone or lime stone and of immature shale, the excavation of the tunnel is commonly associated with a gradual compression of the rock on the both sides of the tunnel, involving a downward movement of the roof. Furthermore, the relatively low resistance against slippage at the boundaries between the so called shale and rock is likely to reduce very considerably the capacity of rock located above the roof to bridge. Hence, in such rock formations, the roof pressure may be as heavy as in a very blocky and seamy rock.

TABLE 2 STRENGTH FACTORS

(*Clause* B-3.1.3)

Category	Strength Grade	Denotation of Rock (Soil)	Unit Weight (kg/m³)	Crushing Strength (kg/cm²)	Strength Factor *f*
I	Highest	Solid, danse quartizite, basalt and other solid rocks of exceptionally high strength.	2 800-3 000	2 000	20
II	Very high	Solid, granite, quartzporphyr, silica shale. Highly resistive sandstones and limestones.	2 600-2 700	1 500	15
III	High	Granite and alike, very resistive sand and limestones. Quartz. Solid conglomerates.	2 500-2 600	1 000	15
IIIa	High	Limestones, weathered granite. Solid sandstone, marble.	2 500	800	8
IV	Moderately strong	Normal sandstone.	2 400	600	6
IVa	Moderately strong	Sandstone shales.	2 300	500	5
V	Medium	Clay-shales. Sand and Limestones of smaller resistance. Loose conglomerates.	2 400-2 600	400	4
Va	Medium	Various shales and slates. Dense marl.	2 400-2 800	300	3
VI	Moderately loose	Loose shale and very loose limestone, gypsum, frozen ground, common marl. Blocky sandstone, cemented gravel and boulders, stoney ground.	2 200-2 600	200-150	2
VIa	Moderately loose	Gravelly ground. Blocky and fizzured shale, compressed boulders and gravel, hard clay.	2 200-2 400	—	1·5
VII	Loose	Dense clay, Cohesive ballast. Clayey ground.	2 000-2 200	—	1·0
VIIa	Loose	Loose loam, loess, gravel.	1 800-2 000	—	0·8
VIII	Soils	Soil with vegetation peat, soft loam, wet sand.	1 600-1 800	—	0·6
IX	Granular soils	Sand, fine gravel, upfill.	1 400-1 600	—	0·5
X	Plastic soils	Silty ground, modified looses and other soils in liquid condition.	—	—	0·3

The load may be taken as uniformly distributed over the diameter of the tunnel.

NOTE — The above is applicable when the distance between the vertex of the pressure parabola from the bottom of the weak layer or from the ground surface is not less than *h*. However, where this condition is not fulfilled the total value of rock load may be assumed.

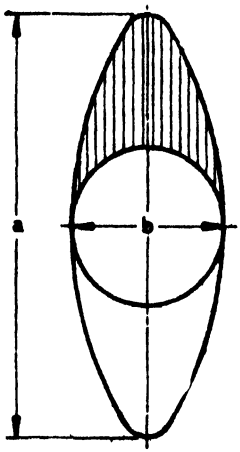

FIG. 4 FENNER'S ELLIPSE

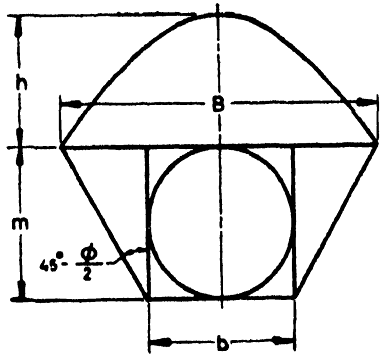

FIG. 5 ASSUMED ROCK LOAD ON A CIRCULAR CAVITY

APPENDIX C

(*Note Under Clause* 7.4.1 and *Clause* 7.4.6.2)

FORMULAE FOR VALUES OF BENDING MOMENTS, NORMAL THRUST, RADIAL SHEAR, HORIZONTAL AND VERTICAL DEFLECTION

C-1. The values of bending moment, normal thrust, radial shear and horizontal and vertical deflection for the loading pattern shown in Fig. 3 are given in Tables 3 to 7.

C-2. For purpose of this appendix, the following notations shall apply:

E = Young's modulus of the lining material,
I = moment of inertia of the section,
K = intensity of lateral triangular load at horizontal diameter,
P = total rock load on mean diameter,
r = internal radius of tunnel,
R = mean radius of tunnel lining,
t = thickness of lining,
W = unit weight of water,
W_c = unit weight of concrete, and
ϕ = angle that the section makes with the vertical diameter at the centre measured from invert.

C-3. For the purpose of this appendix, the following sign conventions shall apply:

a) Positive moment indicates tension on inside face and compression on outside face;

b) Positive thrust means compression on the section;

c) Positive shear means that considering left half of the ring the sum of all the forces on left of the section acts outwards when viewed from inside;

d) Positive horizontal deflection means outward deflection with reference to centre of conduit; and

e) Positive vertical deflection means downward deflection.

TABLE 3 VALUES OF BENDING MOMENTS

(*Clause* C-1)

ϕ	Uniform Vertical Load	Conduit Weight	Contained Water	Lateral Pressure
0	$+0.1250\,PR$	$+0.4406\,W_c\,t\,R^2$	$+0.2203\,W r^2\,R$	$-0.1434\,KR^2$
$\frac{\pi}{4}$	Zero	$-0.0334\,W_c\,t\,R^2$	$-0.0167\,W r^2\,R$	$-0.0084\,KR^2$
$\frac{\pi}{2}$	$-0.1250\,PR$	$-0.3927\,W_c\,t\,R^2$	$-0.1963\,W r^2\,R$	$+0.1653\,KR^2$
$\frac{3\pi}{4}$	Zero	$+0.0334\,W_c\,t\,R^2$	$+0.0167\,W r^2\,R$	$-0.0187\,KR^2$
π	$+0.1250\,PR$	$+0.3148\,W_c\,t\,R^2$	$+0.1724\,W r^2\,R$	$-0.1295\,KR^2$

TABLE 4 VALUES OF NORMAL THRUST

(*Clause* C-1)

ϕ	Uniform Vertical Load	Conduit Weight	Contained Water	Lateral Pressure
0	Zero	$+0.1667\,W_c\,t\,R$	$-1.4166\,W r^2$	$+0.4754\,KR$
$\frac{\pi}{4}$	$+0.2500\,P$	$+1.1332\,W_c\,t\,R$	$-0.7869\,W r^2$	$+0.3058\,KR$
$\frac{\pi}{2}$	$+0.5000\,P$	$+1.5708\,W_c\,t\,R$	$-0.2146\,W r^2$	Zero
$\frac{3\pi}{4}$	$+0.2500\,P$	$+0.4376\,W_c\,t\,R$	$-0.4277\,W r^2$	$+0.2674\,KR$
π	Zero	$-0.1667\,W_c\,t\,R$	$-0.5834\,W r^2$	$+0.3782\,KR$

TABLE 5 VALUES OF RADIAL SHEAR

(*Clause* C-1)

ϕ	Uniform Vertical Load	Conduit Weight	Contained Water	Lateral Pressure
0	Zero	Zero	Zero	Zero
$\frac{\pi}{4}$	$-0{\cdot}250\,0\,P$	$-0{\cdot}897\,6\,W_c\,t\,R$	$-0{\cdot}448\,8\,W\,r^2$	$+0{\cdot}305\,8\,KR$
$\frac{\pi}{2}$	Zero	$+0{\cdot}166\,7\,W_c\,t\,R$	$+0{\cdot}083\,3\,W\,r^2$	$-0{\cdot}024\,6\,KR$
$\frac{3\pi}{4}$	$+0{\cdot}250\,0\,P$	$+0{\cdot}673\,2\,W_c\,t\,R$	$+0{\cdot}336\,6\,W\,r^2$	$-0{\cdot}267\,4\,KR$
π	Zero	Zero	Zero	Zero

TABLE 6 VALUES OF HORIZONTAL DEFLECTION

(*Clause* C-1)

ϕ	Uniform Vertical Load	Conduit Weight	Contained Water	Lateral Pressure
0	Zero	Zero	Zero	Zero
$\frac{\pi}{4}$	$+0{\cdot}014\,73\,\frac{PR^3}{EI}$	$+0{\cdot}050\,40\,\frac{W_c t R^4}{EI}$	$+0{\cdot}025\,20\,\frac{W r^2 R^3}{EI}$	$-0{\cdot}017\,50\,\frac{KR^4}{EI}$
$\frac{\pi}{2}$	$+0{\cdot}041\,67\,\frac{PR^3}{EI}$	$+0{\cdot}130\,90\,\frac{W_c t R^4}{EI}$	$+0{\cdot}065\,45\,\frac{W r^2 R^3}{EI}$	$-0{\cdot}050\,55\,\frac{KR^4}{EI}$
$\frac{3\pi}{4}$	$+0{\cdot}014\,73\,\frac{PR^3}{EI}$	$+0{\cdot}042\,16\,\frac{W_c t R^4}{EI}$	$+0{\cdot}021\,08\,\frac{W r^2 R^3}{EI}$	$-0{\cdot}016\,24\,\frac{KR^4}{EI}$
π	Zero	Zero	Zero	Zero

TABLE 7 VALUES OF VERTICAL DEFLECTION

(*Clause* C-1)

ϕ	Uniform Vertical Load	Conduit Weight	Contained Water	Lateral Pressure
0	Zero	Zero	Zero	Zero
$\frac{\pi}{4}$	$+0{\cdot}026\,94\,\frac{PR^3}{EI}$	$+0{\cdot}092\,79\,\frac{W_c t R^4}{EI}$	$+0{\cdot}046\,40\,\frac{W r^2 R^3}{EI}$	$-0{\cdot}031\,76\,\frac{KR^4}{EI}$
$\frac{\pi}{2}$	$+0{\cdot}041\,67\,\frac{PR^3}{EI}$	$+0{\cdot}139\,17\,\frac{W_c t R^4}{EI}$	$+0{\cdot}069\,58\,\frac{W r^2 R^3}{EI}$	$-0{\cdot}049\,95\,\frac{KR^4}{EI}$
$\frac{3\pi}{4}$	$+0{\cdot}056\,40\,\frac{PR^3}{EI}$	$+0{\cdot}185\,35\,\frac{W_c t R^4}{EI}$	$+0{\cdot}092\,68\,\frac{W r^2 R^3}{EI}$	$-0{\cdot}068\,10\,\frac{KR^4}{EI}$
π	$+0{\cdot}083\,33\,\frac{PR^3}{EI}$	$+0{\cdot}261\,80\,\frac{W_c t R^4}{EI}$	$+0{\cdot}130\,90\,\frac{W r^2 R^3}{EI}$	$-0{\cdot}037\,39\,\frac{KR^4}{EI}$

APPENDIX D

(*Clause* 7.5.1)

BASIC EQUATIONS FOR ANALYSIS OF TUNNEL LINING CONSIDERING IT AND THE SURROUNDING ROCK AS A COMPOSITE CYLINDER

D-1. SCOPE

D-1.1 This appendix contains basic equations for calculating radial and tangential stresses in concrete lining and the surrounding rock mass considering both as parts of a composite cylinder.

D-2. NOTATIONS

D-2.1 For this appendix the following notations shall apply:

P = internal hydrostatic pressure (negative compression);

$\sigma_{t1}, \sigma_{t2}, \sigma_{t3}$ = tangential stress in rock, concrete and steel respectively;

$\sigma_{r1}, \sigma_{r2}, \sigma_{r3}$ = radial stress in rock, concrete and steel respectively;

E, E_2, E_3 = modulus of elasticity of rock, concrete and steel respectively;

m_1, m_2 = poission's ratio of rock, and concrete respectively;

U_1, U_2, U_3 = radial deformation in rock, concrete and steel respectively;

x = radius of element;

B & C, etc = integration constants;

A_s = areas of reinforcement per unit length of tunnel;

a = internal diameter of the tunnel; and

b = external diameter of the lining up to A-line.

D-3. BASIC EQUATIONS

D-3.1 Plain Cement Concrete Lining Considering that it is not Cracked

a) *Basic Equations*:

$$\sigma_r = \frac{m E}{m^2 - 1} \left[B(m+1) - \frac{C}{x^2}(m-1) \right]$$

$$\sigma_t = \frac{m E}{m^2 - 1} \left[B(m+1) + \frac{C}{x^2}(m-1) \right]$$

$$U = Bx + C/x$$

b) *Limit Conditions and Constants*:

1) When $x = \infty$ $\qquad \sigma_{r1} = 0$

2) When $x = b$, $\qquad \sigma_{r1} = \sigma_{r2}$

3) When $x = b$, $\qquad \sigma_{r2} = -p$

4) When $x = b$, $\qquad U_1 = U_2$

D-3.2 Plain Cement Concrete Lining Considering that it is Cracked

a) *Basic Equations for Rock*:

$$\sigma_{r1} = \frac{m_1 E_1}{m_1^2 - 1}\left[B_1 (m_1 + 1) - \frac{C_1}{x^2} (m_1 - 1) \right]$$

$$\sigma_{t1} = \frac{m_1 E_1}{m_1^2 - 1}\left[B_1 (m_1 + 1) - \frac{C_1}{x^2} (m_1 - 1) \right]$$

b) *For Concrete*:

$$\sigma_{r2} = \frac{a. (\sigma_{r2})\ x = a}{x}$$

$\sigma_{t2} = 0$ (since concrete does not take any tangential stress)

c) *Limit Conditions*:

1) When $x = \infty$ $\qquad \sigma_{r1} = 0$

2) When $x = b$, $\qquad \sigma_{r1} = \sigma_{r2}$

3) When $x = a$, $\qquad \sigma_{r2} = -p$

d) Constants are calculated a :

$B_1 = 0$

$$C_1 = \frac{a \cdot b \cdot p\ (m_1 + 1)}{m_1 E_1}$$

$(\sigma_{r2})\ x = a = -p$

D-3.3 Plain Cement Concrete Lining Considering that it is Cracked and Surrounding Rock also is Cracked for a Distance Equal to Radius Beyond which Rock is Massive and Uncracked

a) *For Concrete*:

$$\sigma_{r2} = \frac{a \cdot (\sigma_{r2})_{x=a}}{x}$$

$\sigma_{t2} = 0$

b) *For Cracked Rock*:

$$\sigma_{r1}' = \frac{a. (\sigma_{r2})_{x=a}}{x}$$

$\sigma_{t1}' = 0$

NOTE — Symbol σ_{r1}' and σ_{t1}' refer to cracked zone of rock.

c) *For Surrounding Uncracked Rock*:

$$\sigma_{r1} = \frac{m_1 E_1}{m^2_1 - 1}\left[B_1 (m_1 + 1) - \frac{C_1}{x^2} (m_1 - 1) \right]$$

$$\sigma_{t1} = \frac{m_1 E_1}{m^2_1 - 1}\left[B_1 (m_1 + 1) + \frac{C_1}{x^2} (m_1 - 1) \right]$$

d) *Limit Conditions*:

1) At $x = \infty$ $\quad \sigma_{r1} = 0$
2) At $x = y$, $\quad \sigma_{r1}' = \sigma_{r1}$
3) At $x = b$, $\quad \sigma_{r2} = \sigma_{r1}'$
4) At $x = a$, $\quad \sigma_{r2} = -p$

D-3.4 Reinforced Cement Concrete Lining Considering that it is not Cracked

a) *Basic Equations*:

$$\sigma_r = \frac{m E}{m^2 - 1}\left[B (m + 1) - \frac{C_1}{x^2} (m - 1) \right]$$

$$\sigma_t = \frac{m E}{m^2 - 1}\left[B (m + 1) + \frac{C_1}{x^2} (m - 1) \right]$$

$$U = Bx + o/x$$

$$\sigma_{t3} = \frac{E\,3}{a} \left(B_2 a + \frac{C_2}{a} \right)$$

$$\sigma_{r3} = \frac{E_3 A_s}{a^2}\left(B_2 a + \frac{O_2}{a} \right)$$

b) *Limit Conditions and Constants*:

1) At $x = \infty$ $\quad \sigma_{r1} = 0$
2) At $x = b$, $\quad \sigma_{r1} = \sigma_{r2}$
3) At $x = a$, $\quad \sigma_{r2} - \sigma_{r3} = -p$
4) At $x = b$, $\quad U_1 = U_2$

c) Constants are given by:

$$C_1 = B_2 b^2 + O_2$$

$$C_2 = \left\{ \frac{E_2 m_2 (m_1 + 1)}{E_1 m_1 (m_2 + 1)} \right\} B_2 - \left\{ \frac{E_2 m_2 (m_1 + 1)^2}{E_1 m_1 (m_2 - 1)} \right\} C_2$$

$$-p = B_2 \left\{ \frac{E_2 m_2}{m_2 - 1} - \frac{E_3 A_s}{a} \right\} - \left\{ \frac{E_2 m_2 -}{a^2 (m_2 - 1)} + \frac{E_3 A_s}{a^3} \right\} C_2$$

D-3.5 Reinforced Cement Concrete Lining Considering that it is Cracked and that Because of Radial Cracks it Cannot Take Tangential Tensile Stress

Basic Equations

For rock

$$\sigma_{t1} = \frac{E_1 m_1 C_1}{(m_1 + 1)^2 x^2}$$

$$\sigma_{r1} = -\sigma_{t1}$$

$$U_1 = \frac{C_1}{x}$$

For concrete

$$\sigma_{t1} = 0$$

$$\sigma_{r2} = \frac{a\,(\sigma_{r2})_{x=a}}{x}$$

$$U_2 = \frac{a\,(\sigma_{r2})_{x=a}}{E_2} \cdot \log b/a$$

For Steel

$$\sigma_{t3} = \frac{a.\sigma_{r3}}{A_s}$$

$$\sigma_{r3} = \frac{E_3 A_s}{a^2}\,(aB_2 + C_2/a)$$

$$U_3 = \frac{a^2 \sigma_{r3}}{E_3 A_s}$$

Constants

$$(\sigma_{r2})\,x = a = \frac{-\,pam_1E_1E_2}{am_1E_1E_2 + m_1E_1E_3A_s \log(b/a) + (m_1 + 1)\,E_2E_3A_s}$$

$$C_1 = \frac{-\,ab\,(m_1 + 1)\,(\sigma_{r2})\,x = a}{m_1E_1}$$

$$\sigma_{r3} = (\sigma_{r2})\;x = a + p$$

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CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART V STRUCTURAL DESIGN OF CONCRETE LINING IN SOFT STRATA AND SOILS

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CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART V STRUCTURAL DESIGN OF CONCRETE LINING IN SOFT STRATA AND SOILS

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CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART V STRUCTURAL DESIGN OF CONCRETE LINING IN SOFT STRATA AND SOILS

0. FOREWORD

0.1 This Indian Standard (Part V) was adopted by the Indian Standards Institution on 25 February 1972, after the draft finalized by the Water Conductor Systems Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 Water conductor system occasionally takes the form of tunnels through high ground or mountains, in rugged terrain where the cost of surface pipe line or canal is excessive and elsewhere as convenience and economy dictates. This standard, which is being published in parts, is intended to help the engineers in design of tunnels conveying water. This part lays down the criteria for structural design of concrete lining for tunnels in soft strata and soils, covering recommended methods of design. However, in view of the complex nature of the subject, it is not possible to cover each and every possible situation in the standard and many times a departure from the practice recommended in this standard may be necessary to meet the requirements of a project' and/or site for which discretion of the designer would be required. Some such situations in which special investigations will be required are given below:

a) Where swelling and squeezing types of rocks subject to internal tectonic stresses are met;

b) Where high temperature carbonate formations are met, which may produce carbon dioxide;

c) Where large variations in formation temperatures exist in different sections of a tunnel; and

d) Where anhydrite formations are met which may show expansion sometimes about 30 percent of their volume on becoming wet,

0.3 Other parts of this standard are as follows:

Part I General design,

Part II Geometric design,

Part III Hydraulic design,

Part IV Structural design of concrete lining in rock, and

Part VI Tunnel supports.

0.4 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS : 2-1960*. The number of significant places retained in the rounded off values should be the same as that of the specified value in this standard,

1. SCOPE

1.1 This standard (Part V) covers the criteria for structural design of plain and reinforced concrete lining for tunnels and shafts in soft strata and soils mainly for river valley projects.

NOTE - The provisions may, nevertheless, be used for design of tunnels for roadways, railways, sewage and water supply schemes! provided that all factors peculiar to such projects as may affect the design are taken mto consideration.

1.2 This standard, however, does not cover the design of steel and prestressed concrete lining, the design for concrete lining in swelling and squeezing rocks subject to internal tectonic stresses and design for seismic forces.

2. TERMINOLOGY

2.0 For the purpose of this standard the following definitions shall apply.

2.1 **Soft Strata** — Strata of rocks which are soft either by their nature, usually sedimentary and metamorphic or which have become soft due to alternation and/or shearing, crushing and intensive jointing and which require supports to be installed within a very short period of excavation, but which cannot be easily excavated by hand tools.

2.2 **Soils** — Decomposed and disintegrated rocks which require support immediately after and/or during excavation and can be excavated by hand, tools.

2.3 **Minimum Excavation Line (A-Line)** - A line within which no unexcavated material of any kind and no supports other than permanent structural steel supports shall be permitted to remain.

NOTE — Where due to the nature of strata structural steel supports are essential the minimum excavation line may be at least 75 mm behind the outer flange of the support to accommodate permanent lagging and/or primary-concrete.

*Rules for rounding off numerical values (*revised*).

2.4 Pay Line (B-Line) — An assumed line (beyond A-line) denoting mean line to which payment of excavation and concrete lining is made whether the actual excavation falls inside or outside it.

NOTE — The distance between A and B-lines shall be decided by contracting authority.

2.5 **Primary** Lining — A concrete lining laid j&mediately after excavation and installation of steel supports. This may cover the full section excavated or part section depending on conditions of strata. This may be plain *in situ* concrete or precast concrete segment; or cast iron segments packed with concrete or grout.

2.6 **Final Lining —** It is the concrete between 'primary lining and the finished line of the tunnel.

2.7 Cover — Cover on a tunnel in any direction is the distance from the tunnel profile to the ground surface in that direction. However, where the thickness of the overburden is sizable it's equivalent weight may also be reckoned provided that the rock cover is more than three times the diameter of the tunnel.

3. MATERIALS

3.1 Plain and reinforced concrete shall generally conform to IS : 456-1964*.

4. GENERAL

4.1 The design of tunnel linings requires a thorough study of the geology of the strata to be pierced by the tunnel, the effective cover and a knowledge of the stress strain characteristics, state of stress, etc. It is recommended that a critical study of all these factors be made by test borings, drifts, pilot tunnels or other exploratory techniques. The design of tunnel linings also requires a critical study of the external and internal loading conditions, stresses prior to excavation and their redistribution after excavation, On account of anisotropy of strata and other variables and indeterminate factors, the designer should make a reasonable assessment of the loading conditions taking all the factors into consideration. In soft strata tunnels, besides the above data, it is essential for the designer to have a knowledge of the method of construction which is practicable and economical. The development of loads, in soft strata, is dependent on the size and shape of the tunnel and the methods of construction and time lag between excavation and support. The design should aim at simplicity of construction.

4.2 It is essential for the designer to have a fairly accurate idea of the seepage and the presence or absence of ground water under pressure likely

*Code of practice for plain and reinforced concrete (*second revision*).

to be met with. Where heavy seepage of water is anticipated, the designer shall make provisions for grouting with cement and / or chemicals or extra drainage holes and also consider the feasibility of providing steel lining, if necessary. It is recommended that such designs of alternative use of steel lining be made with the design of plain-reinforced lining, so that the design is readily available should the construction personnel require it, when they meet unanticipated conditions.

4.3 Pressure tunnels with high hydrostatic loads shall have concrete lining reinforced sufficiently to withstand bursting, where inadequate rock cover and unstable ground conditions prevail. Generally, a pressure tunnel should have a reinforced lining if the cover is less than the internal pressure head. In such cases even the provision of a steel plate liner should be considered. If reinforced concrete lining is adopted, the stresses in reinforcement shall be checked to avoid excessive crack width in the concrete as these may lead to seepage from the tunnel into the strata endangering its stability. The final choice would, however, be guided by the geological set up, practicability and economics. The provision of steel liner shall also be considered where high velocity cavitation or erosion of the lining is expected due to high velocity of water.

4.4 Detailed structural analysis and model studies shall be made for design of junctions and transitions for tunnels. Such transitions are difficult to construct in the restricted working space in tunnels and this aspect shall be kept in view so that the proposed structures are easy for construction.

4.5 An adequate amount of both longitudinal and circumferential reinforcement may be provided near the portals of tunnels to resist loads resulting from loosened rock headings or from sloughing of portal cuts. The length to which such reinforcement should be provided depends on the nature of the rock (extent of disintegration, stratification, etc) and the nature and probable behaviour of the overburden near the portal face.

5. LOADING CONDITIONS

5.1 General — The design shall be based on the most adverse combination of probable load conditions, but shall include only those loads which have reasonable probability of simultaneous occurrence.

5.2 Load Conditions — The design loading applicable to tunnel linings shall be classified as normal and extreme design loading conditions. Design shall be made for normal loading conditions (see Appendix A). The design loading shall be as follows:

a) *External Strata Loads (see 7.7)*

b) *Self Load of Lining*

c) *External Water Pressure*

1) *Normal design loading conditions* — The maximum loading obtained from either maximum steady or steady state

condition with loading equal to normal maximum ground water pressure and no internal pressure, or maximum difference in levels between hydraulic gradient in the tunnel, under steady state or static conditions and the maximum downsurge under normal transient operation.

2) *Extreme design loading conditions*-Loading equal to the maximum difference in levels between the hydraulic gradient in the tunnel under static conditions and the maximum downsurge under extreme transient operation or the difference between the hydraulic gradient and the tunnel invert level in case of tunnel empty conditions.

d) *Internal Design Water Pressure (see 7.9)*

1) *Normal design loading conditions* — Maximum static conditions corresponding to maximum water level in the head pond, or loading equal to the difference in levels between the maximum upsurge occurring under normal transient operation and the tunnel centre line.

2) *Extreme design loading conditions* — Loading equal to the difference between the highest level of hydraulic gradient in the tunnel under emergency transient operation and the tunnel centre line.

e) *Seismic Forces — See* Note.

NOTE - According to the prevailing practice the tunnel lining is not designed for seismic forces unless the tunnel crosses an active fault in which case some flexibility is provided at that section to allow for some movement in case of an earthquake. However, at locations where studies indicate that seismic forces will be significant they shall be catered for in the design.

5.3 **Design Loading for Shafts** — The design loading for the shaft walls shall in general be the external earth and ground water pressures. The earth pressure will vary according to the material through which the shaft is excavated and may be computed from the Rankine, Coulomb or slip circle theories in the same way as for retaining walls. This may be considered as uniformly distributed along the perimeter of the shaft except where the material changes in its properties such as in case of steep sloping rock overlain by overburden in which case the differential pressures should be suitably reckoned.

5.3.1 If compressed air sinking is applied, force due to this aspect shall also be taken into account.

5.3.2 If a mantle of thixotropic fluid is used between the walls of shaft and the surrounding soil the resulting reduction in the frictional and external earth pressures with full hydrostatic pressure may be accounted for,

5.3.3 The shaft wall shall also be dimensioned against the axial stresses for tension and bending likely to be encountered during the course of uneven or sudden sinking.

5.3.4 For preliminary estimating the wall thickness may be assumed to be equal to about 8 percent of the shaft diameter.

5.3.5 Circular shafts are preferable as the most straight forward. If other sections (e.g. rectangular) are selected for other considerations, they shall be dealt with as closed frame and corners rounded suitably to reduce concentration of stresses.

5.4 The loading conditions vary from construction stage to operation stage and from operation stage to maintenance stage. The design shall be checked for all probable combinations of loading conditions likely to come on it during all the above stages.

6. STRESSES

6.1 For design of final concrete lining, the thickness of concrete up to A-line shall be considered. Concrete placed as primary concrete will be neglected in the design and its strength can be less than the strength specified for the final lining. The stresses, for concrete and reinforcement shall be in accordance with IS : 456-1964* for design of lining for normal load conditions and shall be increased by 33-1/3 percent for extreme load conditions.

7. DESIGN

7.1 The design of concrete lining of tunnel can only be an intelligent provision for catering to unknown forces and reactions and support conditions. Until reliable data on behaviour of lining is obtained, it is recommended to use approximate methods.

7.2 The elastic behaviour (including flexibility) of the tunnel supports and the primary lining shall be taken into account while designing the lining.

7.3 Design of steel supports, assisted by primary concrete, shall cater for external loads that will develop before final lining is placed. The final lining shall cater for the loads likely to develop after placing of the primary lining and when the work is in operation.

J.4 While designing the final lining the fact that the primary lining and the steel support will also participate in resisting the forces, shall be taken into consideration.

NOTE - To ensure this condition of support the gap between the strata and lining shall be fully closed by grouting and the rock around the tunnel for a distance of at least one diameter shall be strengthened by grouting under pressure.

*Code of practice for plain and reinforced concrete (*second* revision).

7.5 In the case of granular soils and clays, the external load will be taken by the steel supports and primary concrete fully. This lining shall be designed using methods similar to design of culverts on soils. The height of overburden may be the height as calculated by the formula given in Appendix B.

NOTE - It is essential that the gap between strata and the support lining is fully backfilled and grouted at a pressure. not exceeding 2 kg/cm* immediately after the supports and lining are placed,

7.6 The thickness of the lining shall be designed such that the stresses in it are within permissible limits when the most adverse load conditions occur. The minimum thickness of the lining will, however, be governed by requirements of construction. It is recommended that the minimum thickness of plain concrete lining should be 15 cm for manual placement. Where mechanical placement is contemplated the thickness. of the lining at the crown shall be such that the slick line may be easily introduced on the top of the shutter without being obstructed by steel supports. For a 15 cm slick line a clear space of 18 cm is recommended. For reinforced concrete lining, a minimum thickness of 30 cm at the crown is recommended, the reinforcement, however, being arranged in the crown to allow for proper placement of slick line.

7.6.1 For preliminary designs, the thickness of lining may be assumed to be 6 cm/m of the finished diameter of the tunnel in the case of soft strata and 12 cm/m of finished diameter in the case of soils.

7.6.2 Where structural steel supports are used, they shall be considered as reinforcement only, if they can be made effective as reinforcement by use of high tensile bolts/at the joints and/or by proper welding of the joints. A minimum cover of 15 cm shall be provided over the inner flange of steel supports and a minimum cover of 8 cm over the reinforcement bars,

7.7 External Loads from Strata — The determination of the magnitude of the rock load on the supporting structures of the tunnel is a complex problem. This complexity is due to the inherent difficulty of predicting the primary stress conditions in the strata (prior to excavation), and also due to the fact that the magnitude of the secondary pressures developing after the excavation of the cavity, depend on a large number of variables, such as size of cavity, method of excavation, period of time elapsing before the strata is supported, the rigidity of supports, deformation modulus of the surrounding strata, etc.

7.7.1 Secondary external pressure, in general, is understood as the weight of the mass of strata some height above the tunnel which when left unsupported would gradually drop out of the roof. This pressure may develop not only immediately after excavation, but also over period of time after excavation due to adjustment of displacements, in the strata. The

loads are carried both by the tunnel lining and the surrounding strata and this fact shall be considered in design.

7.7.1.1 In the case of granular soils, the loads and side pressures are influenced by the physical properties of the soils. In clays, the water content and plasticity of the clays also affects the pressures on the tunnel linings.

7.7.1.2 In the absence of any data and investigations, it is recommended that the rock loads may be assumed to be acting as uniformly distributed loads and the magnitude assumed as indicated in Appendix B.

7.7.2 *External Pressure of Water*—The lining shall be designed for external water pressure, if any (see 5.2).

7.7.3 *Self Weight of Lining*—The lining will be in close contact with the strata and its weight is distributed over the periphery by frictional forces. However, the weight shall be considered as a uniformly distributed load on the invert (lower half) of the section.

7.7.4 *Weight of Water Contained in the Tunnel* — This shall be considered only for tunnels in soft strata and soils.

7.7.5 *Superimposed Live Loads*—These do not materially affect the tunnels in soft strata- where the diameter of the tunnel is small and the depth of overburden is large. In case of tunnels where the overburden is less, full superimposed load on the basis of normal distribution of loads in foundation strata should be considered in addition to the overburden loads on the tunnel.

NOTE — It may be said that negligible load is transmitted at a depth of more than 3 times the width of the structure causing the superimposed load.

7.7.6 *Side Thrusts or Pressures, Active or Passive* — In the case of tunnels in soft strata, side pressures may exist. The magnitude of these pressures may be estimated on lines similar to the procedure for soils. In case of soils the side pressures may be taken as proportional to the vertical pressures and may be determined by classical theories of soil mechanics. The passive pressures will develop only when there is a deformation. In soft strata and soils, for tunnels constructed with due precautions of grouting around the periphery ensuring a close contact, the passive pressures may be relied upon to bring about a re-distribution of loads.

7.8 Since continuous contact is assumed to be established due to grouting, the strata around and lining both will act and share the loads and deformations Passive pressures may be assumed to be called into play and considered in design.

7.8.1 In the case of tunnels in soft strata and soil, the moments and thrusts may be calculated as indicated in Table 1.

NOTE — The same theory is used for design of culverts.

TABLE 1 CALCULATION OF MOMENTS AND THRUSTS IN CIRCULAR *CULVERTS IN SOIL FOR* $\alpha' = 90°$

(*Clause* 7.8.1)

SL No.	LOADING CONDITION	MOMENT = M NORMAL FORCE = N	$\alpha = 0$ (CROWN) BEDDING*		$\alpha = 45°$ (QUARTER POINT) BEDDING*		$\alpha = 90°$ (SPRINGING) BEDDING*		$\alpha = 135°$ (QUARTER POINT) BEDDING*		$\alpha = 180°$ (BOTTOM) BEDDING*	
			I	II	I	II	I	II	I	II	I	II
i)	Dead load	$M/\rho r^2$ $N/\rho r$	+ 0·3448 − 0·1667	+ 0·2725 0	+ 0·0335 + 0·4375	+ 0·0100 + 0555 4	− 0392 7 + 1·5708	− 0·2983 + 1 ' 5708	− 0035 5 + 1·1334	+ 0·0100 + 1·9696	+ 0·4406 + 1·1667	+ 0·2725 + 2·0000
ii)	Internal water pressure	$M/\gamma_v r^3$ $N/\gamma_v r^2$	+ 0·1724 − 0·5833	-0.1363 − 0·5000	+ 0·0168 − 0·4277	+ 0·0050 − 0·3687	− 0·1964 − 0·2146	− 0149 2 -0.2146	− 0·0168 − 0'7868	+ 0·1363 − 0·3687	+ 0·0050 − 1·4147	+ 0·1363 − 0·5000
iii)	External water pressure	$M/\gamma_v r^3$ $N/\gamma_v r^2$	+ 0·2203 + 0·5833	+ 0·1363 + 1·5000	0·0168 + 1·213 I	+ 0·0050 + 1·6313	− 0·1964 − 1·7854	− 0·1492 + 1·7854	+ 0·0168 + 1·5723	t 0·005 0 + 1·6313	+ 0·1724 + 1·4167	+ 0·1363 + 1·5000
iv]	Excess water pressure in conduit	M	$(P_b - P_k)\, r_b r_k \left[\frac{1}{2} - \frac{r_b r_k}{r_k^2 - r_b^2} \ln \frac{r_k}{r_b} \right]$ throughout the entire ring									
		N	$-P_b r_b$ or $+P_k r_k$ throughout the entire ring									
v)	Uniformly distributed vertical earth pressure	$M/\gamma' r^2 t$ $N/\gamma' r t$	0·2500 0	0.2273 0·0530	0 0·500	− 0·0072 0·5375	-0 2500 1·0000	− 0·2197 1·00	0 0·500	0·014 I 0·7662	0·2500 0	0·1967 0·5836
vi)	Horizontal earth pressure trapezoidal distribution	$M/\gamma' \lambda_a r^2$ $N/\gamma' \lambda_a r$	$(-0·2500t + 0·00417r)$ $(t - 0·375r)$		$(0 - 0·295r)$ $(0·5t - 0·0884r)$		$(0·2500t + 0)$ 0		$(0 + 0·0295r)$ $(0·500t + 0·884r)$		$(-0·2500t - 0·0417r)$ $(t + 0·3750r)$	
vii)	Uniformly distributed horizontal earth pressure	$M/\gamma' \lambda_a r^2 t$ $N/\gamma' \lambda_a r t$	0.25 1·00	0·25 1·00	0 0·50	0 0.50	0·25 0	0·25 0	0 0'50	0 0·50	− 0·25 1·00	− 0·25 1·00

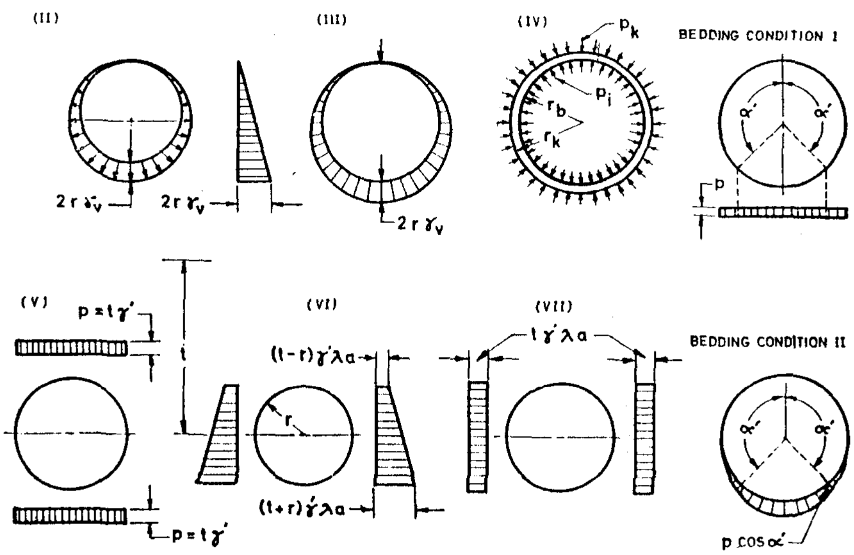
NOTE I -The above formulae for both tedding conditions were also derived by Marquardt for the case of partial embedment.

NOTE 2 — γ' = density of sail

$\lambda_a = \tan^2 (45° - \phi/2)$

ρ = density of concrete

*Please *see* Figures for Bedding Conditions I and II.



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7.8.2 For non-circular lining, it would be necessary to conduct model tests to determine the stress distributions. However, the design may be done assuming uniformly distributed loads as in the case of circular tunnels, and using the same distribution for passive pressures. The design of such indeterminate sections may be done by accepted methods and is not covered by this standard.

7.9 **Design for Internal Waterpressure-The** design for internal water pressure shall be done by considering the lining and surrounding strata, if the strata after grouting is capable of sustaining a part of the internal pressure as a composite thick cylinder. In such a design, the primary concrete may be treated as a part of the thick cylinder.

NOTE-This method suffers from uncertainties of external loads, material properties and indeterminate tectonic forces. In this method the strata surrounding the tunnels is assumed to have reasonably uniform characteristics and strength and that effective pressure grouting has been done to validate the assumption that concrete lining and surrounding strata behave as a composite cylinder. lhe grout fills the cracks and voids in the strata and thus reduces its ability to deform inelastically and increases the modulus of deformation. If the grout pressures are high enough to cause sufficient prestress in the lining the effect of temperature and drying shrinkage and inelastic deformation might be completely counteracted,

7.9.1 For analyzing a circular lining the method given in Appendix C may be adopted. The design shall be such that at no point in the lining and the surrounding rock the stresses exceed the permissible limits.

NOTE — If the rock is not good, tensile stress in concrete may exceed the allowable limit and in such a case, reinforcement may be provided. Reinforcement, however, is not capable of reducing the tensile stresses to a considerable extent. By suitable arrangement, it will help to distribute the cracks on the whole periphery in the form of hair cracks which are not harmful because they may get closed in course of time, or at least they will not result in serious leakage.

7.9.2 For analyzing non-circular linings, the stress pattern may be determined by photo-elastic studies.

8. GROUND WATER DRAINAGE HOLES

8.1 Drainage holes may be often provided in other than water conveying tunnels to relieve external pressure, if any, caused by seepage along the outside of the tunnel lining. It is recommended that drainage holes may be spaced at 6-m centres, at intermediate locations between the grout rings. At successive sections, one vertical hole may be drilled near the crown alternating with two drilled horizontal holes, one in each side wall. Drainage holes shall extend to a minimum of 15 cm beyond the back of the lining or grouted zone. Where suitable, drains encased in suitable graded material, running along the tunnel may be provided by the sides of invert lining with provision of weep holes opening into the tunnel.

8.1.1 In free flowing tunnels drainage holes may be provided above the full supply level. In the case of pressure tunnels, if external water

pressure is substantially more than the internal water pressure, drainge holes may be provided at suitable locations with filters, where necessary, to prevent washing of mountain material into the tunnel. However, when it is not possible to prevent washing of the mountain material into the tunnel drainage holes shall not be provided, if instability is likely to be caused by such washing.

8.1.2 If conveyance of water is through a free pipe located in a tunnel, the horizontal drainage holes shall be drilled near the invert.

9. GROUTING

9.1 Backfill Grouting — Backfill grouting shall be done throughout the length of the concrete lining not earlier than 21 days after the placement of the concrete lining. Stresses likely to develop in concrete at the specified grout pressure may be calculated and seen whether they are within permissible limits depending on the strength attained by concrete by then. Generally the 21 days strength of concrete is sufficient to withstand normal grout pressure which may not exceed about 5 kg/cm².

NOTE - **Backfill** grouting serves to fill voids and cavities between concrete lining and the surrounding strata. This is generally found necessary near the ***crown*** region and may generally extend to not more than 60° angle for circular roof, For flatter arches, the extent may be more.

9.2 **Consolidation Grouting or Pressure Grouting** — Pressure grouting shall be done at a maximum practicable pressure consistent with the strength of lining and safety against uplift of overburden. The depth of grout holes shall be as directed.

NOTE 1 - Pressure grouting consolidates the surrounding strata and fills any gaps caused by shrinkages of concrete. This grouting is normally specified, ***to*** improve yield characteristics and thereby the resistance of strata to carry internal water Pressure. As a rule of thumb a grout ***pressure*** of 1·5 times the internal water pressure in the tunnel may be used subject to the condition that safety against uplift of overburden is ensured. Grout pressures of upto 5 to 10 times the water pressure in the tunnel have been used in some cases.

NOTE 2 — **It is** advantageous to provide a grout curtain by means of extensive deep grouting at the reservoir end of the tunnel to reduce heavy seepage of water and thereby reduce the external water pressure on the lining likely to be developed.

9.3 Grouting shall generally be carried out according to IS: 5878 (Part VII)-1972*.

*Code of practice for construction of tunnels: Part VII Grouting (*under print*).

APPENDIX A

(Clause 5.2)

BASIC CONDITIONS FOR INCLUDING THE EFFECT OF WATER HAMMER IN THE DESIGN

A-l. GENERAL

A-l.1 The basic conditions for including effect of water hammer in the design of tunnels or turbine penstock installations are divided into normal and emergency conditions with suitable factors of safety assigned to each type of operation.

A-2. NORMAL CONDITIONS OF OPERATIONS

A-2.1 The basic conditions to be considered are as follows:

a) Turbine penstock installation may be operated at any head between the maximum and minimum values of forebay water surface elevation.

b) Turbine gates may be moved at any rate of speed by action of the governor head up to a predetermined rate, or at a slower rate by manual control through the auxiliary relay valve.

c) The turbine may be operating at any gate position and be required to add or drop any or all of its load.

d) If the turbine penstock installation is equipped with any of the following pressure controlled devices it will be assumed that these devices are properly adjusted and function in all manner for which the equipment is designed:

1) Surge tanks,
2) Relief valves,,
3) Governor control apparatus,
4) Cushioning stroke device, and
5) Any other pressure control device.

e) Unless the actual turbine characteristics are known, the effective area through the turbine gates during the maximum rate of gate movement will be taken as a linear relation with reference to *time.*

f) The water hammer effects shall be computed on the basis of governor head action for the governor rate which is actually set on the turbine for speed regulation. If the relay valve stops are

adjusted to give a slower governor setting, than that for which the governor is designed this shall be determined prior to proceeding with the design of turbine penstock installation and later adhered to at the power plant so that an economical basis for designing the penstock scroll case, etc, under normal operating conditions can be established.

g) In those instances, where due to higher reservoir elevation, it is necessary to set the stops on the main relay valve for a lower rate of gate movement, water hammer effects will be computed for this slower rate of gate movement also.

h) The reduction in head at various points along the penstock will be computed for rate of gate opening which is actually set in the governor in those cases where it appears that the profile of the penstock is unfavourable. This minimum pressure will then be used as a basis for normal design of the penstock to insure that sub-atmospheric pressures will not cause a penstock failure due to collapse.

j) If a surge is present in the penstock system, the upsurge in the surge tank will be computed for the maximum reservoir level condition for the rejection of the turbine flow which corresponds to the rated output of the generator during the gate traversing time which is actually set on the governor.

k) The downsurge in the surge tank will be computed for minimum reservoir level condition for a load addition from speed-no-load to the full gate position during the gate traversing time which is actually set on the governor.

A-3. EMERGENCY CONDITIONS

A-3.1 The basic conditions to beconsidered as an emergency operation are as follows:

a) The turbine gates may be closed at any time by the action of the governor head, manual control knob with the main relay valve or the emergency solenoid device.

b) The cushioning stroke will be assumed to be inoperative.

c) If a relief valve is present, it will be assumed inoperative.

d) The gate traversing time will be taken as the minimum time for which the governor is designed.

e) The maximum head including water hammer at the turbine and along the length of the penstock will be computed for the

maximum reservoir head condition for final part gate closure to the zero gate position at the maximum governor rate in

$$\frac{2L}{a} \text{ seconds}$$

where

L = the length of penstock, and

a = wave velocity.

f) If a surge tank is present in the penstock system, the upsurge in the tank will be computed for the maximum reservoir head condition for the rejections of full gate turbine flow at the maximum rate for which the governor is designed. The downsurge in the surge tank will be computed for the minimum reservoir head condition for full gate opening from the speed-no-load position at the maximum rate for which 'the governor is designed. In determining the top and bottom elevations of the surge tank nothing will be added to the upsurge and downsurge for this emergency condition of operation.

A-4. EMERGENCY CONDITIONS NOT TO BE CONSIDERED AS A BASIS FOR DESIGN

A-4.1 The other possible emergency conditions of operation are those during which certain pieces of control are assumed to malfunction in the most unfavourable manner. The most severe emergency head rise in a turbine penstock installation occurs from either of the two following conditions of operation:

a) Rapid closure of turbine gates in less than $\frac{2L}{a}$ seconds when the flow of water in the penstock is maximum.

b) Rhythmic opening and closing of the turbine gates when a complete cycle of gate operation is performed in $\frac{4L}{a}$ seconds.

A-4.1.1 Since these conditions of operation require a complete malfunctioning of the governor control apparatus at the most unfavourable moment, the probability of obtaining this type of operation is exceedingly remote, Hence the conditions shall not be used as a basis for design. However, after the design has been established from other considerations it is desirable that the stresses in the turbine scroll case penstock and pressure control devices be not in excess of the ultimate bursting strength or twisting strength of structures for these emergency conditions of operation.

APPENDIX B

(*Clauses 7.5 and 7.7.1.2*)

STRATA LOADS OF TUNNEL LINING

B-1. SCOPE

B-1.1 This appendix gives several alternative methods for evaluating loads from strata on tunnel lining,

B-2. LOAD DISTRIBUTION

B-2.1 The load may be assumed as an equivalent uniformly distributed load over the tunnel soffit over a span equal to the tunnel width or diameter as the case may be.

B-3. LOAD

B-3.1 External loads from the strata may be estimated from the data given in B-3.1.1 to B-3.1.3 for using the appropriate characteristics of the strata.

B-3.1.1 Rock load H_p on the roof of support in tunnel with width B and height H_t, at depth of more than $1{\cdot}5\ (B + H_t)$ may be assumed to be according to Table 2. In case of depths less than $1{\cdot}5\ (B + H_t)$ full may be taken.

TABLE 2 ROCK LOAD ON TUNNELS IN LOOSENING TYPE OF ROCK

Sl No.	Rock Condition	Rock Load H_p m	Remarks
(1)	(2)	(3)	(4)
i)	Hard and intact	Zero	Light lining required only if spalling or popping occurs
ii)	Hard stratified or schistose	0 to $0{\cdot}50\ B$	Light support
iii)	Massive, moderately jointed	0 to $0{\cdot}25\ B$	Load may change erratically from point to point
iv)	Moderately blocky and seamy	$(0{\cdot}25$ to $0{\cdot}35)\ (B + H_t)$	No side pressure

(*Continued*)

TABLE 2 . ROCK LOAD ON TUNNELS IN LOOSENING TYPE OF ROCK — *Contd*

SL No.	ROCK CONDITION	ROCK LOAD H_p m	REMARKS
(1)	(2)	(3)	(4)
v)	Very blocky and seamy	$(0.35 \text{ to } 1.10)(B+H_t)$	Little or no side pressure
vi)	Completely crushed but chemically intact	$1.10\,(B+H_t)$	Considerable side pressure. Softening effect of seepage towards bottom of tunnel. Requires either continuous support for lower ends of ribs or circular ribs
vii)	Squeezing rock	$(1.10 \text{ to } 2.10)(B+H_t)$	Heavy side pressure. Invert struts required
viii)	Squeezing rock, great depth	$(2.10 \text{ to } 4.50)(B+H_t)$	Circular ribs are recommended
ix)	Swelling rock	Up to 80 m irrespective of value of $(B+H_t)$	Circular ribs required, In extreme cases use yeilding support

NOTE 1 — This table has been arrived on the basis of observations and behaviour of supports in Alpine tunnels where the load was designed mainly for loosening type of rock and gives conservative values.

NOTE 2 — The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4 to 6 may be reduced by fifty percent.

NOTE 3 — Some of the most common rock formations contain layers of shale. In an unweathered state, real shales are no worse than other stratified rocks. However, the term shale is often applied to firmly compacted clay sediments which have not yet acquired the properties of rock. Such so called shale may behave in the tunnel like squeezing or even swelling rock.

NOTE 4 - If rock formation consists of sequence of horizontal layers of sand-stone or lime stone and of immature shale, the excavation of the tunnel is commonly associated with a gradual compression of the rock on both sides of the tunnel, involving a downward movement of the roof. Furthermore, the relatively low resistance against slippage at the boundaries between the socalled shale and rock is likely to reduce very considerably the capacity of rock located above the roof to bridge. Hence, in such rock formations, the roof pressure may be as heavy as in a very blocky and seamy rock.

B-3.1.2 The rock load according to the Russian practice depends upon the degree of rock firmness. The strength factors after Protodyakonov are given in Table 3. With cover-depth sufficiently deep for arching action,

the rock load will be defined by the area enclosed by the arch (see Fig. 1) and assumed to act over the diameter of the tunnel.

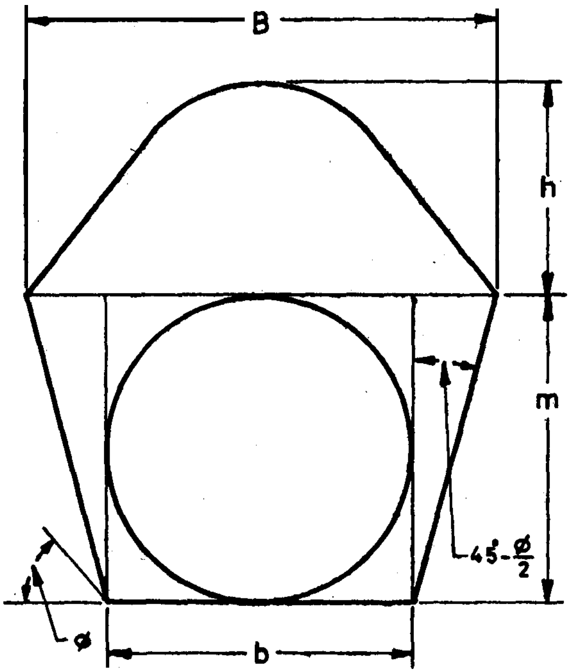

FIG. 1 ASSUMED LOAD ON A CIRCULAR CAVITY

The dimensions of the arch may be obtained from the formulae:

$$h = \frac{B}{2f}$$

$$B = b + 2m \tan (45° - \phi/2)$$

where

f = the strength factor of Protodyakonov (see Table 3),

b = width of tunnel,

m = height of tunnel, and

ϕ = the angle of repose of the soil.

TABLE 3 STRENGTH FACTORS

(*Clause* B-3.1.2)

Category	Strength Grade	Denotation of Rock (Soil)	Unit Weight (kg/cm^3)	Crushing Strength (kg/cm^2)	Strength Factor f
I	Highest	Solid, dense quartzite, basalt and other solid rocks of exceptionally high strength	2 800-3 000	2000	20
II	Very high	Solid granite, quartzporphyr, silica shale, highly resistive sandstones and limestones	2 600-2 700	1500	15
III	High	Granite and alike, very resistive sand and limestones; quartz; solid conglomerates	2 500-2 600	1000	15
IIIa	High	Limestone, weathered granite, solid sandstone, marble	2 500	800	8
IV	Moderately strong	Normal sandstone	2400	600	6
IVa	Moderately strong	Sandstone shales	2300	500	5
V	Medium	Clay-shales, sand and limestones of smaller resistance, loose conglomerates	2 400-2 600	400	4
Va	Medium	Various shales and slates, dense marble	2 400-2 800	300	3
VI	Moderately loose	Loose shale and very loose limestone, gypsum, frozen ground, common marl, blocky sandstone, cemented gravel and boulders, stony ground	2 200-2 600	200-150	2
VIa	Moderately loose	Gravelly ground, blocky and fizzured shale, compressed boulders and gravel, hard clay	2 200-2 400	—	1·5
VII	Loose	Dense clay, cohesive ballast, clayey ground	2000-2200	—	1·0
VIIa	Loose	Loose loam, loose gravel	1800-2 000	—	0·8
VIII	Soils	Soil with vegetation, peat, soft loam, wet sand	1600-1800	—	0·6
IX	Granular soils	Sand, fine gravel, upfill	1 400-1 600	—	0·5
X	Plastic soils	Silty ground, modified loose and other soils in liquid condition	—	—	0·3

In the case of circular tunnels, this can be reduced to:

$$B = d\,[\,1 + 2\tan(45^\circ - \phi/2)\,]$$

$$h = \frac{B}{2f}$$

where

d = diameter of the tunnel.

The load may be taken as uniformly distributed over the diameter of the tunnel.

B-3.1.3 For soils and soft rocks, the unit pressure may be assessed by using Terzaghi's theory:

$$P_v = \frac{B\left[\gamma - \frac{2C}{B}\right]}{2\,K\tan\phi}\left[1 - e^{-K\tan\phi\,\frac{2H}{B}}\right]$$

where

P_v = unit pressure due to load;

$B = 2\left[\frac{b}{2} + m\tan(45^\circ - \phi/2)\right]$;

γ = unit weight of overburden (assume saturated weight if tunnel is in saturated soil);

C = co-efficient of cohesion;

b = width of tunnel (diameter for circular tunnel);

m = height of tunnel (diameter for circular tunnel);

ϕ = angle of internal friction;

K = an emperical factor, which increases from 1·0 for $H = B$, to 1·5 for $H = 2·5\,B$ and beyond; and

H = height of overburden above tunnel crown.

B-4. LATERAL PRESSURE

B-4.1 Lateral pressures in soils may be determined approximately from earth pressure theory as a product of vertical load and earth pressure co-efficient. Some pressures noted indicate that lateral pressures may range from one-fourth to one-third the roof loads.

a) According to Terzaghi a rough estimate of horizontal pressure p_h is given by:

$$p_h = 0{\cdot}3\,\gamma\,(0{\cdot}5\,m + h_p)$$

where

γ = density of soil,

m = height of tunnel section, and

h_p = height of loosening core representing the roof load.

h) In granular soils and rock debris:

$$p_h = \gamma H \tan^2 (45^\circ - \phi/2)$$

where

γ = density of soil,

H = height of overburden above tunnel crown, and

ϕ = angle of internal friction.

c) In cohesive soils the pressure at the crown e^1 and pressure at the invert e^2 may be calculated according to the method* given below:

$$e^1 = h \tan^2 (45^\circ - \phi/2) - 2C \tan (45^\circ - \phi/2)$$

$$e^2 = (h + m \gamma) \tan^2 (45^\circ - \phi/2) - 2C \tan (45^\circ - \phi/2)$$

where

h = height corresponding to vertical load,

ϕ = angle of internal friction,

c = cohesion co-efficient,

m = height of tunnel section, and

γ = density of soil.

d) The lateral active pressure may be increased to take into account the passive resistance developed due to deformation of the lining.

APPENDIX *C*

(*Clause* 7.9.1)

BASIC EQUATIONS FOR ANALYSIS OF TUNNEL LINING CONSIDERING IT AND THE SURROUNDING ROCK AS A COMPOSITE CYLINDER

C-1. SCOPE

C-1.1 This appendix contains basic equations for calculating radial and tangential stresses in concrete lining and the surrounding rock mass considering both as parts of a composite cylinder.

*Soviet practice.

C-2. NOTATIONS

C-2.1 For this appendix the following notations shall apply:

p = internal hydrostatic pressure (negative : compression);

$\sigma_{t_1}, \sigma_{t_2}, \sigma_{t_3}$ = tangential stress in rock, concrete and steel respectively;

$\sigma_{r_1}, \sigma_{r_2}, \sigma_{r_3}$ = radial stress in rock, concrete and steel respectively;

E_1, E_2, E_3 = modulus of elasticity of rock, concrete and steel respectively;

m_1, m_2 = Poisson's ratio of rock and concrete respectively;

U_1, U_2, U_3 = radial deformation in rock concrete and steel respectively;

x = radius of element;

$B, C,$ etc = integration constants;

$A,$ = area of reinforcement per unit length of tunnel;

a = iakrnal diameter of the tunnel; and

b = external diameter of the lining up to A-line.

C-3. BASIC EQUATIONS

C-3.1 Plain cement concrete lining considering that it is not cracked.

$$\sigma_r = \frac{m\,E}{m^2 - 1}\left[B\,(m + 1) - \frac{C}{x^2}(m - 1)\right]$$

$$\sigma_t = \frac{m\,E}{m^2 - 1}\left[B\,(m + 1) + \frac{C}{x^2}(m - 1)\right]$$

$$U = Bx + C/x$$

C-3.13 Limit *Conditions*

a) When $x = \infty, \sigma_{r_1} = 0$

b) When $x = b, \quad \sigma_{r_1} = \sigma_{r_2}$

c) When $x = b, \quad \sigma_{r_2} = -p$

d) When $x = b, U_1 = U_2$

C-3.2 Plain cement concrete lining considering that it is cracked

a) For rock:

$$\sigma_{r_1} = \frac{m_1 E_1}{m_1^2 - 1}\left[B_1 (m_1 + 1) - \frac{C_1}{x^2} (m_1 - 1) \right]$$

$$\sigma_{t_1} = \frac{m_1 E_1}{m_1^2 - 1}\left[B_1 (m_1 + 1) - \frac{C_1}{x^2} (m_1 - 1) \right]$$

b) For concrete:

$$\sigma_{r_2} = \frac{a (\sigma_{r_2})_{x=a}}{x}$$

$\sigma_{t_2} = 0$ (since concrete does not take any tangential stress)

C-3.2.1 *Limit Conditions*

a) When $x = \infty, \sigma_{r_1} = 0$

t

b) When $x = b, \sigma_{r_1} = \sigma_{r_2}$

c) When $x = a$, $\sigma_{r_2} = -p$

C-3.2.1.1 Constants are given by:

$$B_1 = 0$$

$$C_1 = \frac{a.b.p. (m_1 + 1)}{m_1 E_1}$$

$$(\sigma_{r_2})_{x=a} = -P$$

C-3.3 Plain cement concrete lining considering that it is cracked and surrounding rock also is cracked for a distançe equal to a radius y beyond which rock is massive and uncracked

a) For concrete:

$$\sigma_{r_2} = \frac{a (\sigma_{r_2})_{x=a}}{x}$$

$$\sigma_{t_2} = 0$$

b) For cracked rock:

$$\sigma_{r_1}' = \frac{a\,(\sigma_{r_2})_{x=a}}{x}$$

$$\sigma_{t_1}' = 0$$

NOTE — Symbols σ_{r_1}' and σ_{t_1}' refer to cracked zone of rock.

c) For surrounding **uncracked** rock:

$$\sigma_{r_1} = \frac{m_1 E_1}{m_1^2 - 1}\left[B_1\,(m_1 + 1) - \frac{C_1}{x^2}\,(m_1 - 1)\right]$$

$$\sigma_{t_1} = \frac{m_1 E_1}{m_1^2 - 1}\left[B_1\,(m_1 + 1) + \frac{C_1}{x^2}\,(m_1 - 1)\right]$$

C-3.3.1 *Limit Conditions*

a) At $x = \infty$, $\sigma_{r_1} = 0$

b) At $x = y$, $\sigma_{r_1} = \sigma_{r_1}'$

c) At $x = b$, $\sigma_{r_2} = \sigma_{r_1}'$

d) At $x = a$, $\sigma_{r_2} = -p$

C-3.4 Reinforced cement concrete lining considering that it *is* not cracked

$$\sigma_r = \frac{m E}{m^2 - 1}\left[B\,(m + 1) - \frac{C_1}{x^2}(m - 1)\right]$$

$$\sigma_t = \frac{m E}{m^2 - 1}\left[B\,(m + 1) + \frac{C_1}{x^2}(m - 1)\right]$$

$$U = Bx + C/x$$

$$\sigma_{t_3} = \frac{E_3}{a}\left(B_2 a + \frac{C_2}{a}\right)$$

$$\sigma_{r_3} = \frac{E_3 A_s}{a^2}\left(B_2 a + \frac{C_2}{a}\right)$$

C-3.4.1 *Limit Conditions*

a) At $x = \infty$, $\sigma_{r_1} = 0$

b) At $x = b$, $\sigma_{r_1} = \sigma_{r_2}$

c) At $x = a$, $\sigma_{r_2} - \sigma_{r_{r_3}} = -p$

d) At $x = b$, $U_1 = U_2$

C-3.4.1 .1 Constants are given by:

$$C_1 = B_2 b^2 + C_2$$

$$C_1 = \left(\frac{E_2 m_2 (m_1 + 1)}{E_1 m_1 (m_2 + 1)}\right) C_2 - \left(\frac{E_2 m_2 (m_1 + 1)}{E_1 m_1 (m_2 - 1)}\right) B_2 b^2$$

$$-p = B_2\left(\frac{E_2 m_2}{m_2 - 1} - \frac{E_3 A_s}{a}\right) - \left(\frac{E_2 m_2}{a^2 (m_2 + 1)} + \frac{E_3 A_s}{a^3}\right) C_2$$

C-3.5 Reinforced cement concrete lining considering that it is cracked and that because of radial cracks it cannot take tangential tensile stress

a) For rock:

$$\sigma_{t_1} = \frac{E_1 m_1 C_1}{(m_1 + 1)^2 x^2}$$

$$\sigma_{r_1} = - \sigma_{t_1}$$

$$U_1 = \frac{C_1}{x}$$

b) For concrete:

$$\sigma_{t_2} = 0$$

$$\sigma_{r_2} = \frac{a (\sigma_{r_2})_{x = a}}{x}$$

$$U_2 = \frac{a (\sigma_{r_2})_{x = a}}{E_2} \log b/a$$

c) For steel:

$$\sigma_{t_3} = \frac{a . \sigma_{r_3}}{A_s}$$

$$\sigma_{r_3} = \frac{E_3 A_s}{a^2} (aB_2 + C_2/a)$$

$$U_3 = \frac{a^2 \sigma_{r_3}}{E_3 A_s}$$

C-3.5.1 Constants are *given by:*

$$(\sigma_{r_2})_{x = a} = \frac{- pam_1 E_1 E_2}{am_1 E_1 E_2 + m_1 E_1 E_3 A_s \log (b/a) + (m_1 + 1) E_2 E_3 A_s}$$

$$C_1 = \frac{- ab (m_1 + 1) (\sigma_{r_2})_{x = a}}{m_1 E_1}$$

$$\sigma_{r_3} = (\sigma_{r_2})_{x = a} + p$$

AMENDMENT NO. 2 APRIL 2008
TO
IS 4880 (PART 5) : 1972 CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART 5 STRUCTURAL DESIGN OF CONCRETE LINING IN SOFT STRATA AND SOILS

(*Page* 22, *clause* **B-3.1.3**):

a) Substitute '$P_v = \frac{B\left[\gamma - \frac{2C}{B}\right]}{2k \tan\phi}\left[1 - e^{(-2KH/B)\tan\phi}\right]$', *for*

'$P_v = \frac{B\left[\gamma - \frac{2C}{B}\right]}{2K\tan\phi}\left[1 - e^{-K\tan\phi\frac{2H}{B}}\right]$'.

b) Substitute 'C = cohesion' *for* 'C = co-efficient of cohesion'.

[*Page* 23, *clause* **B-4.1**(c)]:

a) Substitute 'e_1' *for* 'e^1' and 'e_2' *for* 'e^2'.

b) Substitute '$e_1 = h\gamma \tan^2(45^\circ - \phi/2) - 2C\tan(45^\circ - \phi/2)$' *for* '$e^1 = h\tan^2(45^\circ - \phi/2) - 2C\tan(45^\circ - \phi/2)$'.

c) Substitute '$e_2 = (h+m)\gamma\tan^2(45^\circ - \phi/2) - 2C\tan(45^\circ - \phi/2)$' *for* '$e^2 = (h + m\gamma)\tan^2(45^\circ - \phi/2) - 2C\tan(45^\circ - \phi/2)$.

d) Substitute 'C = cohesion' *for* 'C = co-efficient of cohesion'.

(WRD 14)

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TO

IS:4880(Part 5)-1972 CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART 5 STRUCTURAL DESIGN OF CONCRETE LINING IN SOFT STRATA AND SOILS

(*Page 24, clause C-2.1, line 7*) - Substitute the following for the existing line:

"m_1, m_2 = Poisson's number of rock and concrete respectively;"

(*Page 24, clause C-2.1, lines 13 and 74*) - Substitute the following for the existing lines:

'a = internal radius of the tunnel; and

b = external radius of the lining up to A-line.'

(BDC 58)

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IS 4880-6 (1971): Code of practice for design of tunnels conveying water, Part 6: Tunnel support [WRD 14: Water Conductor Systems]

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IS : 4880 (Part VI) - 1971
(Reaffirmed 2000)

Indian Standard
CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART VI TUNNEL SUPPORTS

(Second Reprint AUGUST 1990)

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Indian Standard
CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER
PART VI TUNNEL SUPPORTS

0. FOREWORD

0.1 This Indian Standard (Part VI) was adopted by the Indian Standards Institution on 27 October 1971, after the draft finalized by the Water Conductor Systems Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 Very few tunnels are located in perfectly intact strata throughout their whole length, the vast magnitude being driven through rock with defects of one kind or another requiring some support until the permanent lining can be placed. Even intact rock in areas of high initial stresses may require support to prevent popping. Moreover construction of tunnels involves a large number of problems because of the great longitudinal extent of the work and many kinds of conditions are encountered which for maximum economy should be treated differently. In view of this it has been appreciated that it would be futile to prepare a rigid set of rules or procedures which shall be enforced without leaving any latitude for the exercise of discretion by the site engineer. The aim of this standard is to summarize the well-known and proved principles and to describe the commonly used procedures and techniques for providing guidelines which would permit the site engineer to use his discretion.

0.3 In view of the inherent advantages of the steel supports over timber supports, the use of the former is recommended and only steel supports are covered in this standard. In olden days timber was used in tunnel supports but now steel has become almost universally adopted as the standard material for supporting tunnels. Sometimes, however, timber may have to be used for tunnel supports.

0.3.1 Steel supports have the following advantages over timber supports:

a) Steel ribs are easier to handle and require much less storage space;
b) Steel ribs when compared to timber would be smaller, section wise and as such overall cross-sectional area of excavation will be less;
c) Steel ribs become a part of permanent lining and also act as reinforcement. Thus, the thickness of lining will be less;
d) Steel ribs do not deteriorate like timber;

e) Steel ribs can be fabricated to the required shape before hand in the shop and, therefore, their erection is faster; and

f) No specially skilled personnel are required for erection of steel supports.

0.4 This standard is being published in parts. Other parts are as follows:

Part I	General design
Part II	Geometric design
Part III	Hydraulic design
Part IV	Structural design of concrete lining in rock
Part V	Structural design of concrete lining in soft strata and soils

0.5 This standard is one of a series of Indian Standards on tunnels.

0.6 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS : 2-1960*. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

1. SCOPE

1.1 This standard (Part VI) covers the criteria for design of steel supports and roof bolts for tunnels and shafts in rock and soft strata.

NOTE — This standard should be used in conjunction with IS : 5878 (Part IV)-1971†.

2. TERMINOLOGY

2.1 For the purpose of this standard, the definitions given in IS : 5878 (Part IV)-1971† shall apply.

3. MATERIALS

3.1 Structural steel sections for tunnel supports shall confrom to IS : 808-1964‡ and IS : 226-1962§.

3.2 Concrete shall generally conform to IS : 456-1964||.

4. GENERAL

4.1 Before taking up the design of supports, the rock load and pressures likely to act on the supports shall be estimated. The determination of rock

*Rules for rounding off numerical values (*revised*).

†Code of practice for construction of tunnels : Part IV Tunnel supports.

‡Specification for rolled steel beam, channel and angle sections (*revised*).

§Specification for structural steel (standard quality) (*fourth revision*).

||Code of practice for plain and reinforced concrete (*second revision*).

load is a complex problem. This complexity is due to inherent difficulty of predicting the primary stress conditions in the rock mass (prior to excavation) and also due to the fact that the magnitude of the secondary pressure developing after the excavation of the cavity depends on a large number of variables, such as size and shape of cavity, depth of cover, disposition of strike and dip of rock formation in relation to alignment of tunnel, method of excavation, period of time elapsing before rock is supported and the rigidity of supports. These pressures may not develop immediately after excavation but may take a long period after excavation to develop due to adjustment of displacements in the rock mass.

4.1.1 In major tunnels it is recommended that as excavation proceeds, load cell measurements and diametral change measurements are carried out so that rock loads may be correctly estimated. In rocks where the loads and deformation do not attain stable values, it is recommended that pressure measurements should be made by using flat jack or pressure cells.

4.2 In the absence of any data and investigations, rock loads may be estimated in accordance with Appendix B of IS : 4880 (Part V)-1971*.

4.3 As the tunnels generally pass through different types of rock formations, it may be necessary to workout alternative cross-sections of the tunnel depicting other acceptable types of support systems. These types may be selected to match the various methods of attack that may have to be employed to get through the various kinds of rock formations likely to be encountered. 'A' and 'B' lines shall be shown on these sections.

4.4 The support system shall be strong enough to carry the ultimate loads. For a reinforced concrete lining it is economical to consider the supports as an integral part of the permanent lining.

5. TYPES OF STEEL SUPPORT SYSTEM

5.1 Rock tunnel support system of steel may be generally classified into the following principal types:

a) Continuous rib;
b) Rib and post;
c) Rib and wall plate;
d) Rib, wall plate and post; and
e) Full circle rib.

NOTE — Invert strut may be used in addition with types (a) to (d) where mild side pressures are encountered [*see also* IS : 5878 (Part IV)-1971†].

6. SELECTION OF TYPE OF SYSTEM

6.1 For selection of type of support system, a reference may be made to IS : 5878 (Part IV)-1971†.

*Code of practice for design of tunnels conveying water : Part V Structural design of concrete lining in soft strata and soils.

†Code of practice for construction of tunnels : Part IV Tunnel supports.

7. COMPONENTS OF TUNNEL SUPPORT

7.1 For constituents of support system a reference may be made to IS : 5878 (Part IV)-1971*.

8. FACTORS DETERMINING SPACING AND LAYOUT OF SUPPORTS

8.1 The strength and spacing of rib system shall be determined by rock load. For a given rock load and cross-section of tunnel the spacing between the ribs and whether the ribs shall be in two or more pieces shall be worked out. The spacing of the ribs should be so chosen that the sum of the cost of ribs and lagging is minimum. For preliminary designs in ordinary rocks the depth or rib section may be taken as 60 to 75 mm for every 3 metre of bore diameter with ribs spaced at about 1·2 m for moderate loads, 0·6 to 1·0 m for heavy loads and 1·6 m for very light loads.

8.2 For junctions, plugs and control chamber, etc, supports shall be designed to suit the special features of the work and its construction procedures.

8.3 Wooden or concrete blocks of suitable size and thickness may be provided, if necessary, in the bottom portion to provide adequate bearing area to the rib.

8.4 In tunnels, where supports are not to be used as reinforcement, they may be installed plumb or perpendicular to the axis of the tunnel depending on tunnel slope and as found convenient. However, where supports are to be used as reinforcement in pressure tunnels, they may be installed at right angles to the tunnel axis, if practicable.

8.5 For speed of erection of supports it is essential to:

a) design the support system with the minimum number of individual members, consistent with construction convenience;

b) design the joints with utmost simplicity and absolute minimum number of bolts; and

c) fabricate the members with sample bolt and wrench clearances. Time consuming close fits shall be avoided.

9. DESIGN

9.1 General — The design of steel components for tunnel supports shall generally conform to IS : 800-1962†.

9.2 Stresses — Permissible stresses in steel shall be in accordance with IS : 800-1962†. However, if the ribs are bent cold, the maximum permissible fibre stress in steel shall be 1 990 kg/cm².

*Code of practice for construction of tunnels : Part IV Tunnel supports.

†Code of practice for use of structural steel in general building construction (*revised*).

9.3 Ribs — Rock load may be assumed to be transmitted to the ribs at blocking points; each blocking point carrying the load of the mass of rock bounded by four planes, namely, the longitudinal planes passing through mid-points between the blocks and transverse planes passing through mid-points between the ribs to a height equal to the acting rock load. The blocking points may be assumed to be held in equilibrium by forces acting on it in the same manner as panel points in a truss. Values of thrust in the rib may be computed by drawing the force polygon. Ribs shall be designed for the thrust thus computed taking into account the eccentricity of this thrust with reference to the rise of the arc between the blocking points which will cause flexural stresses in addition to direct stresses.

9.4 Tie Rods — *See* IS : 5878 (Part IV)-1971*.

9.5 Lagging — Lagging may be designed for the load of rock mass as shown in Fig. 1 [*see also* IS : 5878 (Part IV)-1971*].

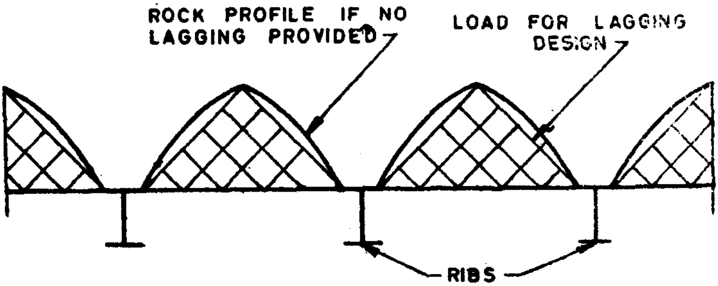

FIG. 1 LOADING DIAGRAM FOR LAGGING

9.6 Liner Plates — Where only liner plates are used for support their cross-sectional area and their joints shall be designed to transmit the thrust (*see* **9.3**). It shall be ensured that liner plates are thoroughly in contact with ground so that passive resistance is developed and no bending moments are induced. For tunnels with more than 3 m diameter liner plates may be reinforced by I-beams. Where liner plates do not form a ring and are used in top half with ribs they shall be designed as lagging [*see also* IS : 5878 (Part IV)-1971*].

9.6.1 The thickness of liner plates may vary from 3 to 10 mm depending upon the size of bore and loads encountered. The size of bolts may vary from 12 to 16 mm diameter.

9.7 Joints — Butt joints should be preferred to spliced joints. In soft grounds and poor rock, welding of joints in the field should be avoided as far as possible.

*Code of practice for construction of tunnels : Part IV Tunnel supports.

10. ROOF BOLTS

10.1 General — Roof bolting follows the principle of fastening the loose rocks near the surface to the solid rock above by means of anchor bolts instead of supporting it from below. Roof bolts not only support the surface rock but also assist it to act as a load carrying element.

10.2 Types — For the types of roof bolts which may be used, a reference may be made to IS : 5878 (Part IV)-1971*.

10.3 Design — Immediately after a tunnel has been advanced by a length t (*see* Fig. 2), the rock in this section expands and settles slightly developing a double arch effect. In the longitudinal direction of the tunnel, the arch rests on the still untouched rock at the front and on the already supported portion at the back (*see* arrows in Fig. 2). The second arch effect, perpendicular to the axis of the tunnel is given by the form of the roof, which usually is an arch in tunnels. The period to which this combined arch will stand without support depends on the geological conditions, the length t and the radius of the tunnel roof, but in most cases, even in badly disintegrated rock, it will be possible to maintain this natural arch for some time, at least a couple of hours. If the natural arch is not supported immediately after mucking, it will continue to sink down slowly until it disintgerates.

10.3.1 The portion that is liable to fall is generally parabolic in cross-section having a depth $t/2$ though the loosening process will never go as deep as this if the movement is stopped by quick support. Nevertheless, it is recommended that the bolts should not be made shorter than t that is twice the depth of the presumed maximum loosening. The natural surrounding rock of the cavity is in this way transformed into a protective arch, the thickness of which is given by the length of the bolts l which should be bigger ~~than~~ t; also $l > \frac{b}{4}$ to $\frac{b}{3}$, as the arch also should have a certain relation to the width of tunnel.

10.3.2 The rock requires a prestress by bolting and the bolts should follow the static principles of prestressing in reinforced concrete as much as possible. As it is not possible to place bolts in the way of stress bars at the lower side of a beam, they should at least be given an oblique position in order to take the place of bent-up bars and stirrups (*see* Fig. 3).

10.3.2.1 With an arch instead of a beam, the shear forces will be greatly reduced by the vault effect but even in arch shaped roofs, shear forces may be caused by joint systems, especially by system of parallel layers like sedimentary formations, scists, etc. Hence the bolts should not only be made to exert a strong prestress to the rock but also should be set in a direction which

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*Code of practice for construction of tunnels : Part IV Tunnel supports.

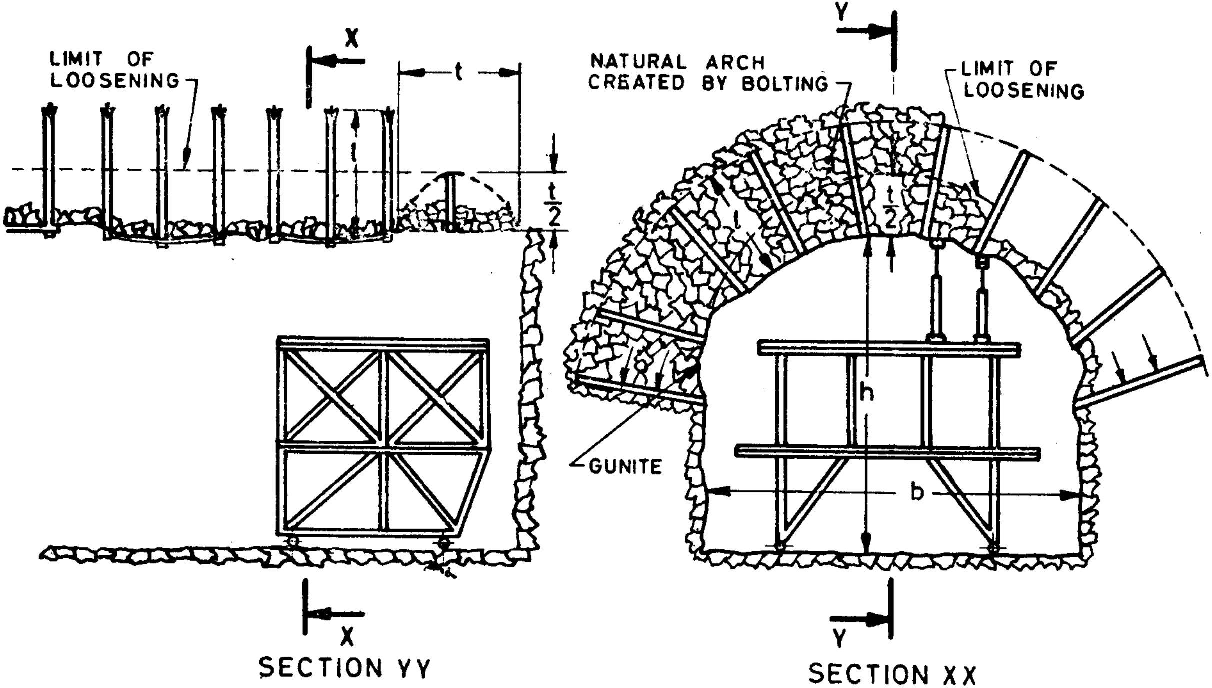


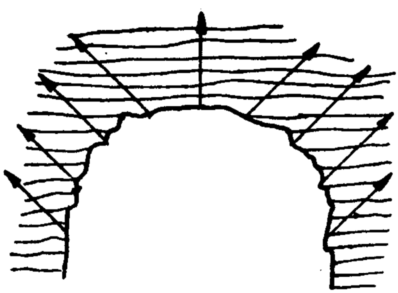
Fig. 2 Diagrammatic Sections Demonstrating Principles of Rooi Bolting

suits best to the static demands of the geological conditions as shown in Fig. 3.

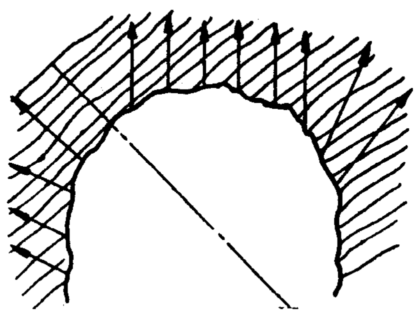
10.3.2.2 Just as a static member of reinforced concrete has to be prestressed before receiving the load, the rock also shall be prestressed by bolting before the load develops. This means that the space '*t*' in Fig. 2 shall be bolted immediately after blasting and at the same time as the next round is being drilled.

10.3.3 The space between the bolts shall be chosen in accordance with the length and diameter of the bolts. Assuming for instance that a bolt is given a load of 12 tonnes taking into consideration a sufficient factor of safety, this would correspond to a rock volume of 4·5 m³ at a density of 2·65 t/m^3. If the length of the bolt allows a thickness of the protective arch of 2·5 m each bolt will correspondingly carry an area of the load of:

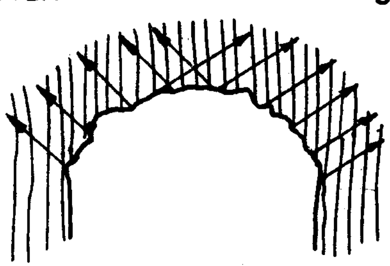
$$\frac{12}{2{\cdot}65 \times 2{\cdot}5} = 1{\cdot}8 \text{ m}^2$$



3A HORIZONTAL JOINT SYSTEM



3B INCLINED JOINT SYSTEM



3C VERTICAL JOINT SYSTEM

Fig. 3 Roof Bolting in Strata Running at Various Angles of Dip

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"ज्ञान एक ऐसा खजाना है जो कभी चुराया नहीं जा सकता है"
Bhartṛhari—Nītiśatakam
"Knowledge is such a treasure which cannot be stolen"

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IS : 4880 (Part VII) - 1975
(Reaffirmed 2000)

Indian Standard

CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART VII STRUCTURAL DESIGN OF STEEL LINING

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CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART VII STRUCTURAL DESIGN OF STEEL LINING

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Indian Standard

CODE OF PRACTICE FOR DESIGN OF TUNNELS CONVEYING WATER

PART VII STRUCTURAL DESIGN OF STEEL LINING

0. FOREWORD

0.1 This Indian Standard was adopted by the Indian Standards Institution on 27 May 1975, after the draft finalized by the Water Conductor Systems Sectional Committee had been approved by the Civil Engineering Division Council.

0.2 The type of lining chosen for tunnels depends upon the rock properties and rock cover over the tunnel and the type of tunnel. If the tunnel has to withstand very high pressures, it will have to be steel lined. There is, so far, no generally accepted method for the design of steel lining. The purpose of this code is to outline the criteria to be adopted in the design. The tendency of modern designers, however, is to avoid any steel lining. Rock conditions will have to be determined for taking a final decision as to when and where steel lining becomes indispensable.

0.3 Steel lining is a steel plate provided to inner surface of water carrying tunnel for any or all of the following purposes:

a) To resist bursting pressure of water carried by the tunnel;

b) To prevent water losses;

c) To provide protection from seepage of water from the surrounding mass like rock, concrete, etc; and

d) To provide a smooth surface for flow of water.

0.4 This standard is being published in parts. Other parts of this standard are as follows:

Part I	General design
Part II	Geometric design
Part III	Hydraulic design
Part IV	Structural design of concrete lining in rock
Part V	Structural design of concrete lining in soft strata and soils
Part VI	Tunnel supports

0.5 This standard is one of a series of Indian Standards on tunnels.

0.6 For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS: 2-1960*. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

1. SCOPE

1.1 This standard (Part VII) covers the general requirements and design of steel lining of tunnels for conveyance of water from reservoirs to hydraulic turbines in hydro-power plants or *vice versa* in case of reversible pump turbines in pumped storage schemes or for other similar installations.

1.2 This Code does not cover the design of specials like manifolds, wye-piece, transitions, etc.

2. GENERAL

2.1 Dimensions and Shape

2.1.1 The circular section of the tunnel is most suitable for lining from point of view of design and fabrication. Steel liner in rectangular shape is generally not provided.

2.1.2 The most economical dimensions and shape of the pressure conduit to be lined should be decided on the basis of the economic studies. The particular size for which the sum of the value of power annually lost and the annual charges is at a minimum, is selected. Provisions of **2.3.1.1** of IS : 4880 (Part III)-1968† are applicable in this connection.

2.2 Contraction and Expansion — To minimize head losses and to avoid cavitation tendencies along the tunnel surface contraction and expansion transitions shall be designed in accordance with IS : 4880 (Part III)-1968†.

2.3 Limiting Velocity — The velocities in a steel lined tunnel depend on the economic considerations and governing conditions of turbine. These velocities may range between 1·6 to 9 m/s.

NOTE — Low velocities normally occur when steel lining is provided in shorter lengths in a concrete lined tunnel where reducing the diameter may not be practicable or desirable.

*Rules for rounding off numerical values (*revised*).
†Code of practice for design of tunnels conveying water: Part III Hydraulic design.

2.4 Loss of Head — The loss of head due to different reasons shall be worked out conforming to IS : 4880 (Part III)-1968*.

3. DESIGN

3.1 Material — The permissible stresses to be adopted depend upon the quality of steel to be used. For satisfactory welding, the plates shall be of fire box quality conforming to IS : 2002-1962† and IS : 2041-1962‡.

3.2 Design Stresses and Factor of Safety — The design stresses and factor of safety to be adopted depend upon the yield point stress of steel and location where steel lining is to be provided.

3.2.1 For steel lining to be provided in tunnels where sharing of internal load by rock is not to be taken into account the design stress to be adopted shall be equal to the yield point stress with a factor of safety of 1·67.

3.2.2 In case of pressure tunnels passing through rock and requiring steel lining it is a general practice to assume that the rock shares the internal water pressure to some extent depending upon its quality and rock cover. Wherever rock share is considered, the allowable design stress for free shell shall not exceed 0·67 to 0·9 times yield stress of steel depending upon the type of rock and available rock cover. The rock participation shall be in accordance with **5.1.2**.

3.2.3 For emergency conditions expected to exist for short period of time and at infrequent intervals, design stress equal to yield point stress of steel with factor of safety of one shall be adopted.

3.3 Joint Efficiency — Joint efficiency or weld factor assumed for purpose of design varies for different kinds of joints and different methods of inspection and testing. The joint efficiency also varies for different types of steel.

3.3.1 For all longitudinal welded joints if completely radiographed or ultrasonically tested and weld defects repaired, the joint efficiency is taken as 100 percent otherwise the efficiency is taken as 90 percent.

3.4 For all specials like manifolds, transitions, etc, and lining thickness exceeding 38 mm, stress relieving shall be done.

4. DESIGN LOADS

4.1 Internal Pressure — The steel lining of a tunnel is designed for maximum internal pressure. The maximum internal pressure is the load on the pipe per unit area caused by the maximum static water head and

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*Code of practice for design of tunnels conveying water: Part III Hydraulic design.
†Specification for steel plates for boilers.
‡Specification for steel plates for pressure vessels.

the anticipated increase in the static head due to water hammer effect development when arresting or releasing the flow of water. The water hammer effect may be determined using the details given in the 'draft Indian Standard Criteria for water hammer analysis' (*under preparation*).

NOTE — Until the above standard is published, the matter shall be subject to agreement between the concerned parties.

4.2 External Pressure — The steel lining shall be designed for the external water pressure which is either the difference between the ground level vertically above the tunnel and the tunnel invert level, or the maximum level from which water is likely to find its way around the steel lining. The liner should also be checked against grouting pressure during construction.

4.3 Longitudinal Stresses Caused by Radial Strain — Radial expansion of steel caused by internal pressure tends to cause longitudinal contraction with corresponding tensile stress equal to 0·303 of hoop tension in circular lining. However, this is generally negligible.

5. THICKNESS OF LINING

5.1 The thickness of steel lining in tunnel depends upon:

a) minimum handling thickness,

b) that required for internal pressure after rock participation, and

c) that required for external pressure.

5.1.1 Regardless of pressure conditions, a minimum handling thickness is recommended for the shell to provide the rigidity required during fabrication. This is given by the formula:

$$t_{min} = \frac{d + 50}{400}$$

where

t_{min} = minimum handling thickness in cm, and

d = internal dia of shell in cm.

5.1.2 For internal pressure, the shearing between rock and liner depends upon the modulus of elasticity of rock. The rock modulus may be determined by *in situ* tests and percentage of rock participation can then be calculated from **5.1.2.1**.

5.1.2.1 The rock participation may be worked out from the following formula:

$$\lambda = \frac{\sigma_t - \frac{E.r_o}{r}}{\sigma_t - P\left[\frac{E}{E_r}(1+\mu) + \frac{E}{2E_c} \times \frac{r}{r_c}(r_e^2 - r^2) + \frac{E}{2E_r} \times \frac{r^2}{r_e d}(d^2 - r_e^2)\right]}$$

where

λ = proportion of internal pressure transferred to rock;

σ_t = allowable stress in steel liner;

E = modulus of elasticity of steel;

r_o = initial gap between liner and concrete caused by shrinkage and creep of concrete and temperature effect;

r = inside radius of steel liner;

P = internal pressure in tunnel;

E_r = modulus of elasticity of rock;

μ = Poisson's ratio of rock;

E_c = modulus of elasticity of concrete;

r_e = outside radius of concrete lining; and

d = radius to the end of radial fissures in rock, that is, where the natural compressive stresses in the rock are just exceeded by the tensile stresses caused by internal pressure.

5.1.3 The thickness of steel lining should be determined for hoop stresses and checked for combined hoop and longitudinal stresses.

5.2 The thickness of circular steel lining for internal pressure should be worked out by formula:

$$t = \frac{p.R}{s.e}$$

where

t = shell thickness in cm,

p = maximum internal pressure after rock participation in kg/cm^2,

R = inner radius of finished surface of tunnel in cm,

s = design stress in kg/cm^2, and

e = efficiency of longitudinal joint to be in accordance with **3.3**.

5.3 Shell Thickness Due to External Water Pressure — As regards to steel lining provided in tunnels the question of designing the steel shell for external pressure arises when water pressure develops around steel shell. Due to plastic yield in the rock, which may continue for considerable time, it is possible that a gap may be developed between steel and concrete or rock and concrete after tunnel is emptied releasing internal pressure. The gap between the liner and concrete or concrete and rock also depends on the initial gap due to shrinkage of concrete, temperature variation and plastic deformation of liner concrete and rock. Even though the rock would be grouted for some depth all around the periphery of the tunnel, the probability of water seeping through and accumulating in the cavity around the steel lining cannot be ruled out. Such water will be under pressure and would cause external pressure on steel lining. The shell thickness should be designed for external pressure or anchors can be provided.

5.4 Designing the Shell for External Pressure — The design thickness of the shell is worked out by the formulae given at **5.4.1** and **5.4.2.** However, a factor of safety of at least 1·5 shall be allowed over the calculated critical external pressure. Permissible critical external pressure can be increased by providing suitable anchors on the liner.

5.4.1 Barrot's formula:

$$13\,\frac{P_c^2}{EK^2}+2P_c\left(1+\frac{K}{2}+\frac{6\,J}{KR^1}-\frac{F_e}{KE}\right)-\left(2\,F_eK-\frac{EK^2J}{R^1}-\frac{F_e^2}{E}\right)=0$$

where

P_c = critical external pressure in kg/cm²,

E = modulus of elasticity in kg/cm²,

$K = \frac{t}{R^1}$,

J = gap between steel lining and concrete in cm,

R^1 = external radius of curvature in cm,

F_e = yield point stress in kg/cm², and

t = thickness of shell in cm.

5.4.2 Amstutz's equation:

$$\left(\frac{f_n}{E'}+\frac{J}{R^1}\right)\left[1+3K^2\frac{f_n}{E'}\right]^3$$

$$=1{\cdot}68\,K\frac{f'_y-f_n}{E'}\left[1-\frac{K}{4}\cdot\frac{f'_y-f_n}{E'}\right] \quad \text{......(1) and}$$

$$1-\frac{PK}{2f_n}=0{\cdot}175\frac{K}{E'}(f'_y-f_n) \quad \text{..........(2)}$$

where

f_a = allowable stress in material,

$E' = \dfrac{E}{1 - \mu^2}$,

J = gap between steel liner and concrete,

E = Young's modulus of elasticity,

R^1 = external radius of curvature in cm,

$K = \dfrac{t}{R^1}$,

t = thickness of shell in cm,

$f'_y = \dfrac{f_y}{\sqrt{1 - \mu - \mu^2}}$,

P = critical external pressure in kg/cm^2,

f_y = yield stress of material, and

μ = Poisson's ratio.

5.5 Protection — Inner surface of the steel lining shall be provided with adequate protection against rusting by applying suitable paint.

6. DESIGN OF ANCHORS/STIFFNER RINGS FOR RESISTING EXTERNAL PRESSURE

6.1 In the present context anchors have a restricted role of resisting external pressures only. The anchors provided can be continuous along the circumference and placed at certain intervals. These can be made of either angle irons or latticed structure depending on the strength required. Stiffening provided should be simple and should not interfere with construction and proper placing and compaction of concrete. Contact between the concrete and the steel lining should be complete. However, the liner thickness may be designed to resist the external pressures and not to provide any anchors or stiffener rings, since these hinder proper concreting and leave pockets behind the liner.

6.1.1 Spacing of anchors along the periphery of circular steel lining of a given thickness can be worked out by Backte's formula. The formula is as follows:

$$L = 2a.R$$

$$a = \frac{(F_{e1})^{3/2} \,.\, E \,.\, t^4}{P_c^{5/2}\; R^4}$$

where

L = spacing of anchors/stiffner rings in cm,

F_{e1} = stress at lower yield point of steel in kg/cm^2,

E = modulus of elasticity of steel in kg/cm^2,

t = thickness of shell in cm,

P_e = critical external pressure on the lining in kg/cm^2, and

R = internal finished radius of tunnel in cm.

6.1.2 There is no method by which anchor/stiffner ring spacing in the longitudinal direction can be determined. Hence, this is also kept the same as that along the circumference.

6.2 Size of Anchors/Stiffner Rings

6.2.1 The function of anchor/stiffner ring is to sew the steel lining into concrete and thereby increase the number of bulges the steel lining shell should take due to external pressure.

6.2.2 Anchors/stiffner rings should therefore be strong enough not to be sheared off, should the steel lining shell try to slip along the concrete and should not allow all the inward bulges to concentrate at a single location.

6.2.3 As a rough guide the anchors should be 25 to 40 mm in diameter and about 300 to 500 mm in length.

7. BANDS OVER STEEL LINING SHELLS

7.1 It may not be possible to provide steel lining for the designed thickness according to **6** due to availability of steel plates of required thickness or limitations due to fabrication difficulties. Steel bands are provided to share the load in steel shell, under such circumstances. Bands can also be used with advantage for strengthening the fabricated steel lining when rock at a particular location is found to be weaker than anticipated, after excavation.

8. CHANGE IN THE THICKNESS OF STEEL LINING

8.1 The thickness of steel lining at different locations depends upon various factors enumerated in **6.1** to **6.4**. Especially when the pressure tunnel or shaft passes through different layers of the rock, such as good medium or bad, the steel thickness provided in bad or medium layer of the rock is extended into next better layer of the rock for a length equal at least to one diameter of the pressure tunnel or shaft. Beyond this the thickness is reduced in steps of not more than 5 mm each till the smaller thickness required for better rock is reached. These steps are kept at individual shells of fabricated smallest length. When the lining emerges from the tunnel it should be designed for full internal water pressure and due care should be taken of stress concentrations occurring in the surrounding rock.

INTERNATIONAL SYSTEM OF UNITS (SI UNITS)

Base Units

Quantity	*Unit*	*Symbol*
Length	metre	m
Mass	kilogram	kg
Time	second	s
Electric current	ampere	A
Thermodynamic temperature	kelvin	K
Luminous intensity	candela	cd
Amount of substance	mole	mol

Supplementary Units

Quantity	*Unit*	*Symbol*
Plane angle	radian	rad
Solid angle	steradian	sr

Derived Units

Quantity	*Unit*	*Symbol*	*Conversion*
Force	newton	N	$1\ \mathrm{N} = 1\ \mathrm{kg.1\ m/s^2}$
Energy	joule	J	$1\ \mathrm{J} = 1\ \mathrm{N.m}$
Power	watt	W	$1\ \mathrm{W} = 1\ \mathrm{J/s}$
Flux	weber	Wb	$1\ \mathrm{Wb} = 1\ \mathrm{V.s}$
Flux density	tesla	T	$1\ \mathrm{T} = 1\ \mathrm{Wb/m^2}$
Frequency	hertz	Hz	$1\ \mathrm{Hz} = 1\ \mathrm{c/s}\ (\mathrm{s^{-1}})$
Electric conductance	siemens	S	$1\ \mathrm{S} = 1\ \mathrm{A/V}$
Pressure, stress	pascal	Pa	$1\ \mathrm{Pa} = 1\ \mathrm{N/m^2}$

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